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NOTICE

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FHWA COTR: Heather Shelsta

FHWA Technical Working Group Leader: Barry Siel, PE; Silas Nichols, PE; Scott Anderson, PhD, PE; Brian Lawrence, PE

Contractor Technical Consultants: Jie Han, PhD, PE

This manual is the updated version of FHWA NHI-06-019/20, prepared by Ryan R. Berg & Associates, Inc.; authored by V. Elias, J. Welsh, J. Warren, R. Lukas, J. Collin and R. Berg; FHWA Technical Consultants J. DiMaggio and S. Nichols.

16. Abstract

This FHWA Geotechnical Engineering Circular No. 13 provides guidance on Ground Modification Methods, and also serves as the reference manual for FHWA NHI courses No. 132034, 132034A, and 132034B on Ground Modification Methods. The purpose of this manual is to introduce available ground modification methods and applications to design generalists, design specialists, construction engineers, and specification and contracting specialists involved with projects having problematic site conditions.

An introductory chapter provides a description, history, functions, and categories of ground modification. A description of the web-based *GeoTechTools* (http://www.geotechtools.org) technology selection guidance system and geotechnology catalog is also provided in the first chapter. The introductory chapter is followed by stand-alone technical category chapters. Each category chapter includes a broad introduction to the technical category including typical applications, a listing of common technologies used in the U.S., and summaries for specific technologies in the category. Each technology summary includes: description; advantages and limitations; applicability; complementary technologies; construction methods and materials; photographs; design guidance; quality assurance methods; costs; specifications; and reference list. Each technical category and the technology summaries therein reflect current practice in design, construction, contracting methods, and quality procedures. This publication was prepared with the practicing transportation specialist in mind and with the benefit of extensive industry review.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

PREFACE

One of the major tasks within geotechnical engineering is to design, implement and evaluate ground modification schemes for infrastructure projects. During the last forty years significant new technologies and methods have been developed and implemented to assist the geotechnical specialist in providing cost-effective solutions for construction on marginal or difficult sites.

The impetus for ground modification has been both the increasing need to use marginal sites for new construction purposes and to mitigate risk of failure or of poor performance. During the past several decades, ground modification has come of age and reached a high level of acceptance in the geotechnical community. Its use is now routinely considered on most projects where poor or unstable soils are encountered. From the geotechnical engineer's point of view, ground modification means the modification of one or more of the relevant design engineering properties (e.g., increase in soil shear strength, reduction of soil compressibility, and reduction of soil permeability) – or the transfer of load to more competent support layers. From the contractor’s point of view, ground modification may mean a reduction in construction time and/or a reduction in construction costs. Both points of view are valid reasons to consider the use of ground modification techniques and are often mutually inclusive.

Herein, ground modification is defined as the alteration of site foundation conditions or project earth structures to provide better performance under design and/or operational loading conditions. Ground modification objectives can be achieved using a large variety of geotechnical construction methods or technologies that alter and improve poor ground conditions where traditional over-excavation and replacement is not feasible for environmental, technical or economic reasons. Ground modification has one or more of the following primary functions, to:

- increase shear strength and bearing resistance,
- increase density,
- decrease permeability,
- control deformations (settlement, heave, distortions),
- improve drainage,
- accelerate consolidation,
- decrease imposed loads,
- provide lateral stability,

- increase resistance to liquefaction, and/or
- transfer embankment loads to more competent subsurface layers.

The purpose of GEC 13 is to introduce available ground modification methods and applications to design generalists (i.e., project planners, roadway designers, consultant reviewers, etc.), design specialists (i.e., geotechnical, structural, pavement, etc.), construction engineers, specification writers, and contracting specialists involved with projects having problematic site conditions. This publication was prepared with practicing transportation specialists and generalists in mind.

The introductory chapter provides a description, history, functions, and categories of ground modification. Additionally, the role of ground modification in addressing project risks and constraints and risk mitigation, and contracting mechanisms and their impact on selection of ground modification technologies are described. The chapter also includes description of the web-based *GeoTechTools* (http://www.geotechtools.org) technology selection guidance system, and its use for the initial screening process of developing a short-list of technologies applicable to a given project. The *GeoTechTools* geotechnology catalog, of over 50 technologies, and the engineering tools provided for each technology are described. A discussion of final project-specific technology selection that extends beyond the initial screening that can be developed within *GeoTechTools* is included in Chapter 1. Through incorporation of technology and project specific factors, a 12-step process is presented that leads to selection of a preferred, specific technology for a given project.

The introductory chapter is followed by stand-alone technical category chapters. Each category chapter includes a broad introduction to the technical category including typical applications, a listing of common technologies used in the United States, and summaries for specific technologies in the category. Each technology summary includes: description; advantages and limitations; applicability; complementary technologies; construction methods and materials; design guidance; quality assurance methods; costs; specifications; and reference list. Each technical category and the technology summaries therein reflect current practice in design, construction, contracting methods, and quality assurance procedures. Transportation focused case histories are included for select technologies.

This 2016 GEC 13 reference manual on Ground Modification Methods is an update to the 2006 FHWA-NHI-06-019/020 Ground Improvement Methods reference manual. Lead author of the 2006 manual was Victor Elias, PE, and is his last major work. Mr. Elias had a distinguished professional career and provided significant contributions to the design and construction of safe, cost-effective geotechnical works in transportation works. He had been the Principal Investigator for several major research and/or implementation projects focused

on durability of soil reinforcement materials, design guidance and specifications for retaining walls foundations and, and ground improvement methods.

In addition to the contributions of Victor Elias, the co-authors of this GEC 13 recognize the efforts of Barry Siel, Silas Nichols, Scott Anderson, and Brian Lawrence of the FHWA. Their input and guidance into this update, and the previous works have been invaluable. The input received from industry review was very insightful and beneficial. The co-authors thank Harlee Drury for drafting new and revised figures, and thank Sue Stokke and Pete Hunsinger of Iowa State University's Institute for Transportation (InTrans) for their meticulous 508-compliance work with the Word and pdf files.

Chapters and technology categories contained in this Volume II of the FHWA Ground Modification reference manual set:

Chapters and technology categories contained in the companion Volume I of the FHWA Ground Modification reference manual set:

Chapter 6

COLUMN-SUPPORTED EMBANKMENTS

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1.0 DESCRIPTION AND HISTORY

The problems associated with constructing highway embankments over soft compressible soils (e.g., large settlements, embankment instability, and the long period of time required for consolidation of the foundation soil) have led to the development and extensive use of many of the ground modification techniques in use today. Prefabricated vertical drains (PVDs), surcharge loading, geosynthetic reinforcement, stone columns, deep soil mixing, and lightweight fill have all been used to solve the settlement and stability issues associated with construction of embankments on marginal soils. However, when time constraints are critical to the success of the project, owners have resorted to another innovative approach: column supported embankments (CSE) with or without a geosynthetic reinforced load transfer platform (LTP). In the last 25 years, this technology has been used successfully by over a dozen state DOTs.

1.1 Description

1.1.1 Column-supported Embankments

CSEs consist of stiff vertical columns that are designed to transfer the load of the embankment through the soft compressible soil layer to a firm foundation. Selection of the type of column used for the CSE will depend on the design loads, constructability of the column, cost, etc., and will be discussed in more detail in Sections 2 and 3. The load from the embankment must be effectively transferred to the columns to prevent punching of the columns through the embankment fill causing differential settlement at the surface of the embankment. If the columns are placed close enough together, soil arching will occur and the full embankment load will be transferred to the columns. A CSE is illustrated in Figure 6-1.

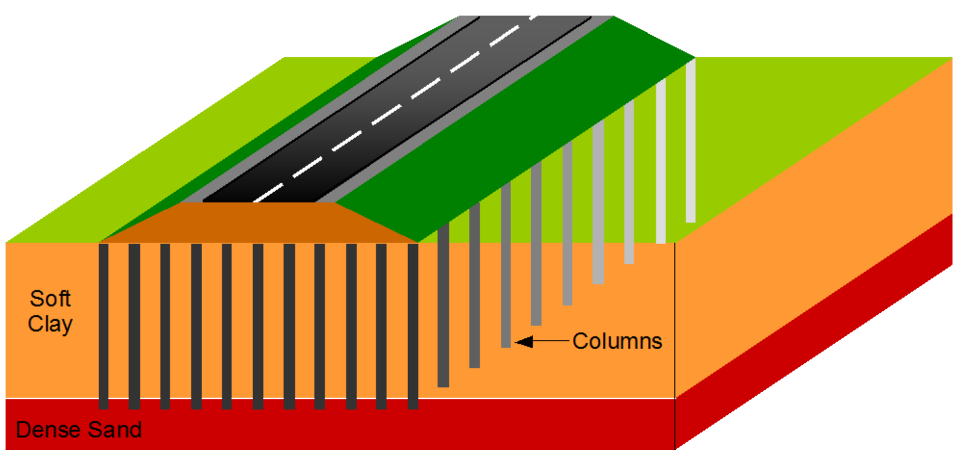

Figure 6-1. Column-supported embankment.

The columns in Figure 6-1 are spaced relatively close together (i.e., 4 to 6 feet), and some battered columns may be required at the sides of the embankment to prevent lateral spreading. In order to significantly reduce the number of columns required to support the embankment and increase the efficiency of the design, a load transfer platform (LTP) either geosynthetically reinforced or with no reinforcement may be used. A CSE with geosynthetic reinforcement is schematically shown in Figure 6-2.

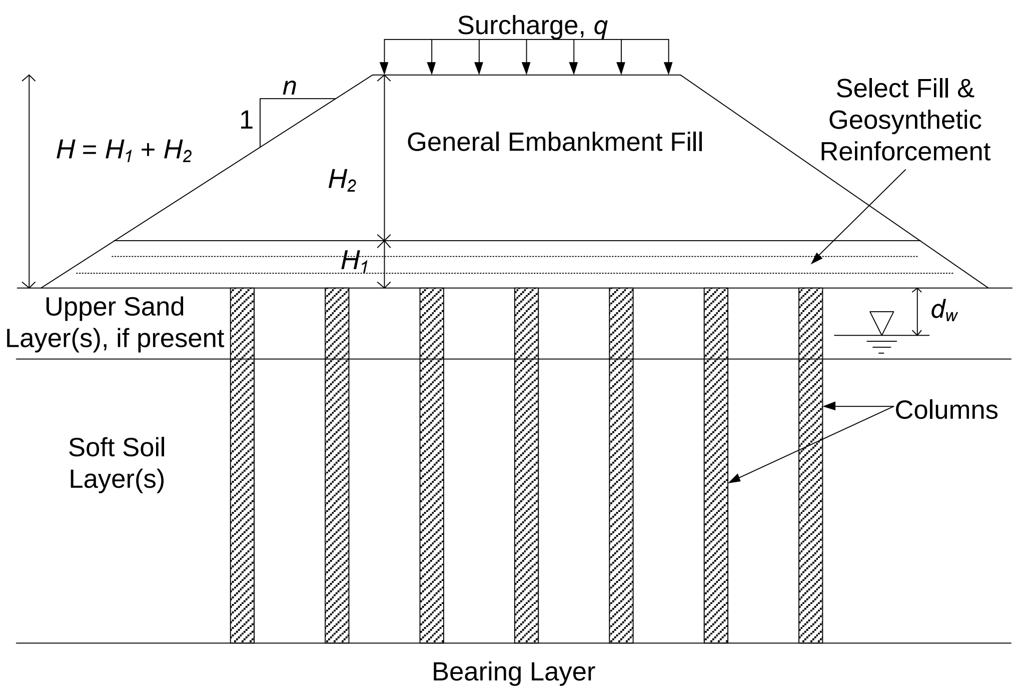

Figure 6-2. Column-supported embankment with geosynthetic reinforcement.

1.1.2 Support Columns

The support columns that are used with this technology include steel H-piles, steel pipe piles, auger cast piles, precast concrete piles, and timber piles. Conventional steel and concrete piles often provide higher axial load capacity than is required for CSEs and are, therefore, less economically attractive compared to timber piles and newer formed-in-place column types.

The newer formed-in-place column types that have been used for columns in CSEs include: soil mix columns, aggregate columns, and cement based columns. These columns are discussed in the Chapter 7 Soil Mixing and Chapter 5 Aggregate Columns. The selection of the column will depend on the design loads, foundation support layer, any stiff intermediate layers that need to be penetrated, special equipment requirements, speed of installation, and local availability and cost of the columns. The requirements and selection of the columns will be covered in detail in Sections 2 and 3. It is important to note here that the technology is not

dependent on any one column type, thereby allowing the contractor to select the most economical column based on the design and performance requirements established for the project by the specifying agency.

1.1.3 Load Transfer Platform

The load transfer platform (LTP) is used to efficiently transfer the embankment or structure load to the columns without allowing unacceptable deformations to occur between columns that would reflect to the surface of the embankment. Three types of load transfer platforms are available. A reinforced concrete structural mat may be used to transfer the embankment load to the columns. This requires a structural design of the mat to assure that the load is effectively transferred to the columns. Concrete mats have generally been found to be economically cost prohibitive and will not be discussed further in this chapter.

The second and third types of LTPs consist of select granular structural fill either reinforced with one or more layers of geosynthetic, or without reinforcement. The remainder of this chapter will focus on the design and construction of granular LTPs. The design of the load transfer platform will be covered in detail in Section 4. Currently, there are two fundamental approaches to geosynthetic reinforced LTPs: the catenary method and the beam method. The catenary method considers the reinforcement to act as one layer at the interface between the subgrade and columns and the embankment. Select fill may or may not be used above the geosynthetic and the geosynthetic acts as a catenary. The beam method considers multiple (i.e., 3 or more) layers of reinforcement spaced vertically, typically 8 to 16 inches apart within the LTP to create a beam of reinforced soil.

1.2 Historical Overview

The first documented use, for a highway application, of CSE with geosynthetic reinforcement was in 1984 for a bridge approach embankment in Europe (Reid and Buchanan 1984). Concrete piles were used as the columns for the project. Each column had a reinforced concrete pile cap. The clear span between pile caps varied from 6.6 to10 feet. One layer of geosynthetic reinforcement was used to create the load transfer platform. The height of the embankment was 30 feet.

The first application of CSE with geosynthetic reinforcement in the United States was in 1994 for the Westway Terminal in Philadelphia, PA. This project involved the support of a large diameter tank for the storage of molasses. The foundation consisted of vibro-concrete columns (VCC) and an LTP, and is shown in Figure 6-3.

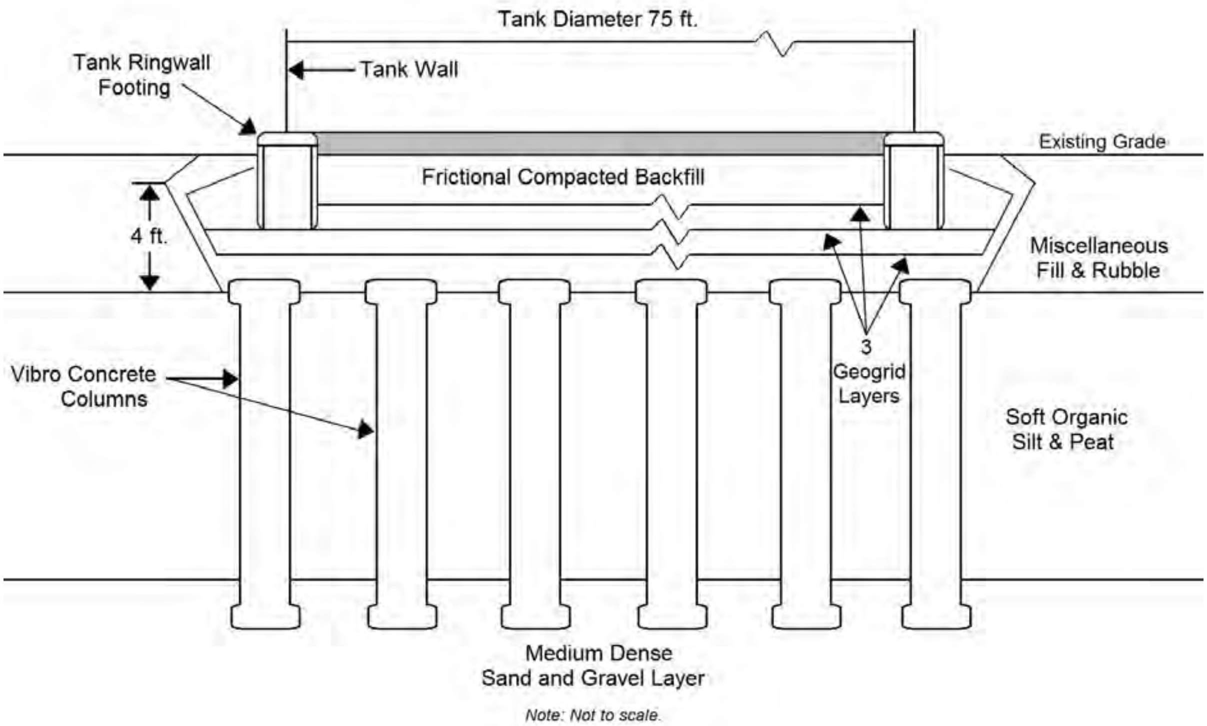

Figure 6-3. Westway terminal project.

The LTP consisted of a well graded granular fill, reinforced with three layers of geogrid reinforcement. The CSE was selected over a more conventional pile foundation with a concrete mat because of both time and cost savings.

One of the first (2001) transportation-related projects in the United States to use CSE was for an embankment over soft soils, at a river crossing, for the New Jersey Light Rail (Young et al. 2003). The foundation for the embankment consisted of VCC and an LTP. The VCCs were placed on a 6.6 to 10 feet center-to-center triangular spacing. The LTP was 3 feet thick, and was reinforced with three layers of geogrid. A well-graded granular soil was used as structural fill for the LTP. A typical cross-section of the project is shown in Figure 6-4. The CSE was selected for this project to eliminate the “bump” at the end of the bridge without having to wait for the foundation soil to consolidate.

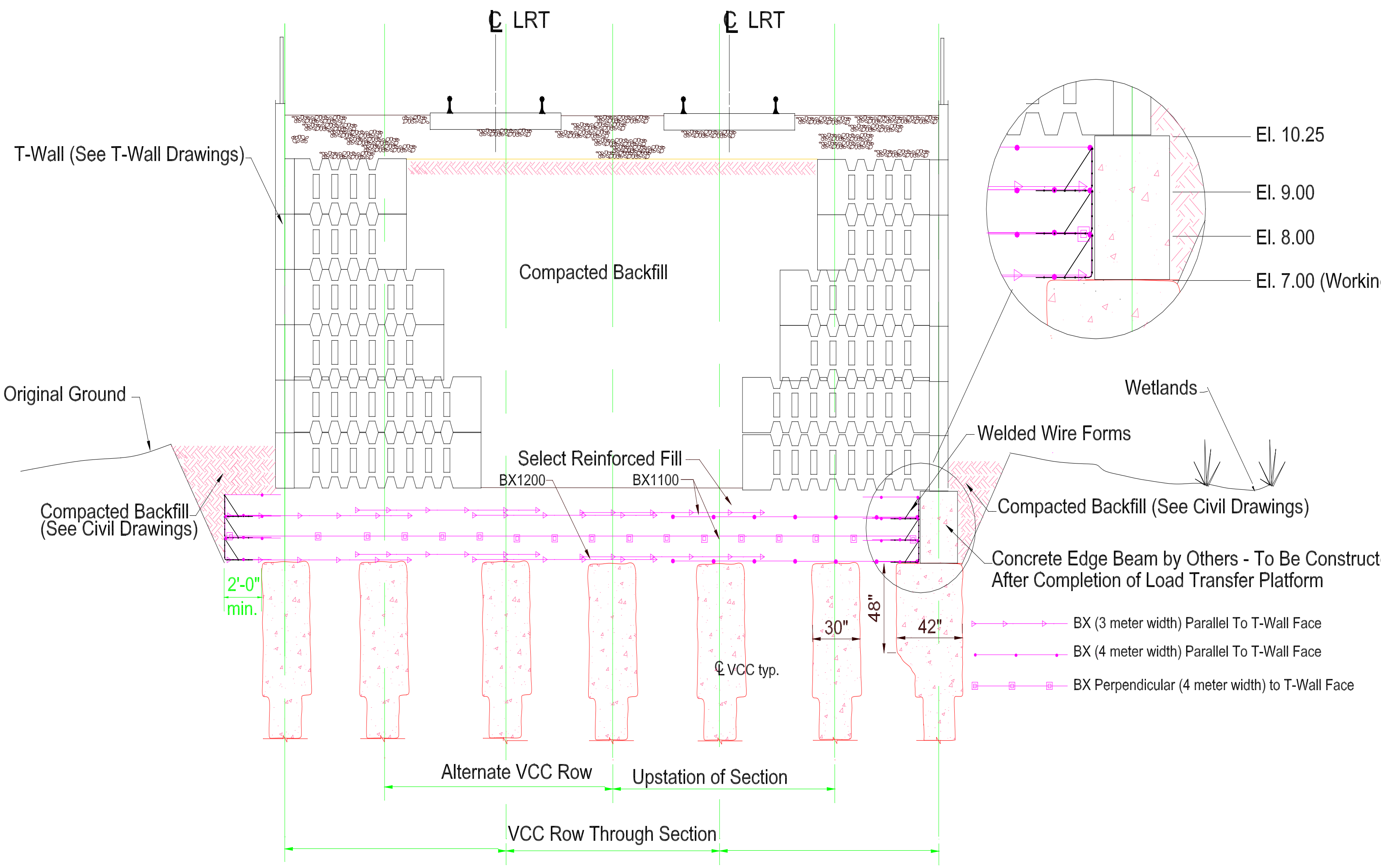

Figure 6-4. New Jersey Light Rail project.

The use of CSE with geosynthetic reinforcement has increased dramatically in the past twenty years both in the United States and abroad. More than 20 case histories were available in the literature documenting the use of this technology in 1999 (Han 1999). Additional case histories are now available in the literature.

In the United States, the design of CSE has advanced significantly in the last decade. The early design method predominately used was the beam method. The previous version of this manual provided specific design guidance for this approach. This method is a semi-empirical method that was developed based on laboratory testing and observation of performance of full-scale structures. This design method was instrumental in the advancement of this technology as it was a relatively straightforward design method. However, as with most emerging technologies, more refined design methods have been developed. Today, the beam design approach is being replaced by the use of a more analytically correct method that is based on load and displacement compatibility, which we will refer to as the load and displacement compatibility (LDC) method (Sloan et al. 2013). The LDC method is discussed in detail in Section 4. Alternatively, some specialty contractors and geotechnical consulting firms are developing CSE designs by performing deformation analyses using 2D and 3D numerical modeling.

1.3 Focus and Scope

The focus and scope of this chapter on CSEs is to identify problems that have been successfully solved by the use of CSE and to synthesize the current state-of-the-practice of CSE construction and design. In addition, this chapter will provide guidance on the selection process for when and where to use CSEs. References are cited where more detailed technical information can be obtained, and typical costs are given in order to make a preliminary technical and economic evaluation regarding whether CSE can solve a specific problem. The intent of this document is to serve as a reference on CSEs and how they may solve a specific problem by discussing their construction, utilization, and limitations.

1.4 Glossary

A variety of terms are used with reinforced soil technologies. For clarity, they are defined as throughout this manual, as follows:

Aggregate columns are stone columns and rammed aggregate piers capable of supporting 25 to 150 kips of vertical load.

Column Supported Embankment (CSE) consists of stiff vertical columns that are designed to transfer the load of the embankment through the soft compressible soil layer to a firm foundation.

Cement based columns use Portland cement binder with aggregate for column construction, and are more rigid than an aggregate column. Cement binder with aggregate can be used to construct a cemented aggregate column. Another cement based column option it use concrete for construction of the columns, such as vibro-concrete columns (VCCs), controlled modulus columns (CMCs), and continuous flight auger (CFA) piles.

Driven pile columns are traditional or conventional piles, such as steel H, steel pipe, or timber, which are used to support an embankment.

Load and Displacement Compatibility (LDC) is the CSE design methodology recommended within this chapter.

Load Transfer Platform (LTP) consists of select granular structural fill either reinforced with one or more layers of geosynthetic, or without reinforcement, that transfers the embankment or structure load to the columns without allowing unacceptable deformations to occur between columns that would reflect to the surface of the embankment.

Geosynthetics is a generic term that encompasses flexible polymeric materials used in geotechnical engineering such as geogrids, geotextiles, and geostraps.

Prefabricated Vertical Drain (PVD) is band shaped (rectangular cross-section) product consisting of a geotextile jacket surrounding a plastic core. Water flows from soil through the filter into the core of the drain and from there upwards to the soil surface.

Reinforcement is used only for those *inclusions* where soil-inclusion stress transfer occurs continuously along the inclusion, (i.e., a soil reinforcement).

Vibro-Concrete Columns (VCC) are considered a related technology to stone columns, with concrete replacing the stone in the column.

1.5 Primary References

The primary references for this chapter are listed below:

- Collin, J.G. (2007). U.S. State-of-Practice for the Design of the Geosynthetic Reinforced Load Transfer Platform in Column Supported Embankments. *Soil Improvement*, Schaefer, V.R., Filz, G.M, Gallagher, P.M., Sehn, A.L., and Wissmann, K.J., Editors, Geotechnical Special Publication No.172, Geo-Institute of ASCE, Reston, VA.
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V.R., Filz, G.M, Gallagher, P.M., Sehn, A.L., and Wissmann, K.J., Editors, Geotechnical Special Publication No. 172, Geo-Institute of ASCE, Reston, VA.

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- Sloan, J.A., Filz, G.M., and Collin, J.G. (2013). *Columns Supported Embankments: Field Tests and Design Recommendations.* Center for Geotechnical Practice, Virginia Polytechnic Institute and State University, Blacksburg, VA.

2.0 FEASIBILITY CONSIDERATIONS

This section discusses applications, advantages and disadvantages, feasibility evaluations, limitations, and alternative solutions for the CSE technology.

2.1 Applications

CSEs have traditionally been used to support embankments over soft soil when time is not available to allow consolidation of the soft foundation soil when using PVDs and surcharge loads, or when differential or total settlement and overall stability are a concern. The main purpose of a CSE is to transfer the embankment loads through the columns to a competent soil or rock layer beneath the soft foundation soil. Applications where CSE technology is appropriate for transportation include:

- embankment stabilization
- roadway widening
- bridge approach fill stabilization
- bridge abutment, and other foundation support

Other applications that have utilized this technology include foundation support for storage tanks, commercial office building foundation support (i.e., shallow foundations supported on a CSE), and retaining wall foundation support. The database of successful projects continues to expand, and with the development of new, more cost effective column systems and improved design tools CSE use will continue to grow.

One typical application of CSE technology is the stabilization of large area loads, such as highway embankments. The use of CSEs offers a practical alternative, where conventional embankments cannot be constructed due to stability, time, or environmental considerations. Applications include moderate to high fills on soft soils, and embankment fills that may be contained by Mechanically Stabilized Earth (MSE) retaining walls.

A considerable amount of highway widening and reconstruction work will be required in future years. Some of this work will involve building additional lanes immediately adjacent to existing highways constructed on moderate to high fills over soft cohesive soils, such as those found in wetland areas. For this application, differential settlement between the existing and new construction is an important consideration, in addition to embankment stability. Support of the new fill on CSE offers a viable design approach to mitigate such differential settlement.

CSE can be used to support bridge approach fills, to provide stability, and to reduce the costly maintenance problem from settlement at the joint between the approach fill and bridge. In 2001, the New Jersey Light Rail used a CSE for the approach embankment for a river crossing. One side of the embankment was contained by a modular concrete retaining wall system, and the other side of the embankment sloped downward to the adjacent grade. The CSE included the use of VCC as the columns and three layers of geosynthetic reinforcement to create the LTP (Young et al. 2003) to eliminate the *bump* at the end of the bridge.

Under favorable conditions, CSEs can be constructed to greater heights than a conventional approach embankment over soft foundation soils. Therefore, the potential exists to reduce the length of bridge structures by extending the approach fills. Embankment fills can be placed more quickly, due to the fact that the embankment places little load on the soft foundation soil.

CSEs can be used to support bridge abutments at sites that are not capable of supporting abutments on conventional shallow foundations. At such sites, an important additional application involves the use of MSE walls supported on CSEs. CSEs have been used successfully to support building foundations when located in areas that contain soft compressible foundation soils.

2.2 Advantages and Disadvantages of CSE

2.2.1 Advantages

CSEs provide a technical and potentially economical alternative to more conventional construction techniques (i.e., surcharge loading and PVDs, staged construction with or without geosynthetic reinforcement). The key advantage to CSE is that construction may proceed rapidly in one stage. There is no waiting time for dissipation of pore water pressure in the soft foundation soil. CSEs are also more economical than the removal and replacement of deep deposits of soft soils, particularly on larger sites where the groundwater is close to the surface. Where the infrastructure precludes high-vibration techniques, the type of column used for the CSE system may be selected to minimize or eliminate the potential for vibrations. Total and differential settlement of the embankment may be drastically reduced when using CSE over conventional approaches. CSEs may also be considered as a sustainable alternative to other ground improvement methods (e.g., CSEs may be less energy intensive than constructing and removing a temporary surcharge).

One major benefit of CSE technology is that it is not limited to any one column type. If contaminated soils are anticipated at a site, the column type may be selected so that there are no spoils from the installation process. If very soft soil is anticipated, cement based columns,

auger cast-in-place piles or timber piles may be selected as the column type for the project. In stronger foundation soils, stone columns or rammed aggregate piers may be economically more attractive. The designer has the flexibility to select the most appropriate column for the project.

2.2.2 Potential Disadvantages

A major disadvantage of CSE is often high initial construction cost when compared to other solutions. However, if the time savings and reduced maintenance due to improved long-term performance when using CSE technology are included in the economic analysis, the cost may be far less than other solutions.

2.3 Feasibility Evaluations

CSE may be used whenever an embankment must be constructed on soft compressible soil. To date, the technology has been limited to embankment heights less than about 50 feet. The depth of the soft soil layer is typically not a critical component in the determination of feasibility because of the many different types of columns available for use to obtain bearing in a firm layer below the soft layer.

A generalized summary of the factors that should be considered when assessing the feasibility of utilizing CSE technology on a project are presented below:

1. Typically the preliminary spacing of the columns has been limited so that the area replacement ratio is between 3.5 to 10%, however on some projects it has been as low as 2.5%. The area replacement ratio is the ratio of the plan view cross-sectional area of the column to the plan view cross-sectional area of the unit cell, which is the area of influence for each column. However, if column caps are used, the area replacement ration should be determined based on the cross-sectional area of the column cap. Refer to Section 4.2 for details. This recommendation is based on the empirical performance of documented case histories of CSEs.
2. The embankment height should be greater than the critical height. The critical height is the minimum height at which there is no practically significant differential settlement at the surface of the embankment. The width of the column, or pile cap if included, and spacing between columns significantly influence the critical height. Refer to Section 4.5 for the recommended method to estimate the critical height.
3. The fill required to create the LTP shall be a select structural fill with an effective friction angle greater than or equal to 35°.
4. The columns shall be designed to carry the entire load of the embankment.

5. The clear span between columns should be less than or equal to 10 feet.
6. CSE technology reduces post construction settlements of the embankment surface to typically less than 2 to 4 inches and differential settlement to less than 1-inch.

2.3.1 Geotechnical Considerations

The key geotechnical considerations when evaluating the feasibility of CSE for a project are associated with the following:

1. Does the soft compressible soil extend to grade so that a working platform will be required prior to installing the columns?
2. If soft soils do not extend to grade, are the surface soils adequate with regard to strength, stiffness, and layer thickness to act as the LTP or assist the LTP in distributing the load from the embankment to the columns?
3. Are there intermediate stratums of relatively stiff/dense soils that cannot be penetrated by the columns?

The above items are factors that will affect the overall economy of the system but are not considered to be deal breakers for the use of the technology.

2.3.2 Environmental Considerations

The selection of the most appropriate column system should consider the environmental effects of the installation. For example, if stone columns are being considered for a project, vibro-replacement stone columns are traditionally jetted in place, thus removing the finer portions of the influenced soil. The resulting fines-laden jetted water has to be temporarily contained to allow for sediment deposition and disposal. Jurisdictions have varying regulations regarding the processes for these operations. Also, unknown contaminants may be removed and transferred to the environment by the jetting water. The designer may select an alternate column system that does not replace the in situ soils (i.e., dry vibro-displacement stone columns, cement based columns, etc.).

In urban environments where noise and vibrations may be unacceptable, appropriate columns may be selected accordingly.

2.3.3 Site Consideration

Site conditions should always be considered when selecting a ground improvement technology. This technology may be used on sites with limited headroom as the type of column may be changed to suit the site conditions. There are not many site constraints that this technology cannot accommodate. However, an important consideration in the use of this

technology is the thickness of the LTP. If, for example, an MSE wall is to be placed on top of the LTP and the leveling pad for the MSE wall is 2 feet below existing grade, and the water table is 3 feet below existing grade, a 4 foot thick LTP would be difficult to construct as it would require 3 feet of excavation below the groundwater table.

2.4 Limitations

The major limitation of this technology is that for very low height embankments on soft soil projects where the soft soils start at the ground surface, the columns may need to be so close together to satisfy the critical height design requirement, that the CSE system becomes uneconomical.

2.5 Alternative Improvement Methods

Alternate ground improvement systems that should be considered when evaluating CSE include surcharge pre-loading with or without PVDs, staged construction with or without geosynthetic reinforcement, lightweight fill, and combinations of these technologies. The chapters on Vertical Drains and Accelerated Consolidation; Lightweight Fills; and Reinforced Soil Structures should be reviewed for more information on these alternate systems. Additionally, designers should also consider using a bridge structure as an alternative to an embankment when crossing soft compressible soil sites.

3.0 MATERIALS AND CONSTRUCTION

3.1 Columns

3.1.1 Materials

The columns are an integral part of CSEs, and many types of columns are available to the designer. Driven pile columns (i.e., timber, steel H, steel pipe, pre-cast concrete, cast-in-place concrete shell, and shells driven without mandrel) may be used. Driven piles are generally considered to be very stiff columns with a modulus of elasticity between 1,000 to 30,000 ksi (modulus for timber piles is 1,000 ksi). The load carrying capacity of driven piles may be calculated in accordance with GEC 12 (2016).

Another column option is continuous flight auger (CFA) piles. CFA piles use concrete for construction of the columns and, therefore, are considered to be stiff columns. The load carrying capacity of these piles may be calculated in accordance with GEC 8 (2007). Settlement of these types of columns is typically governed by the capacity of the foundation soil.

In addition to CFA piles, there are a variety of other cement based column technologies that are related, and similar, to aggregate columns. Cement binder with aggregate can be used to construct a cemented aggregate column, and Portland cement concrete can be used to construct columns. Many of these are proprietary technologies developed by ground modification contractors. Some are equipment and installation variations, and may be more suited to specific installation conditions, such as beneath the water table or in very soft soils. See *GeoTechTool*s, and ground modification contractor websites, for information on cement based columns.

Aggregate columns (stone columns and rammed aggregate piers) have modulus values between 5 to 9 ksi, which is considerably lower stiffness than driven pile columns or cement based columns. The design of these columns is presented in the Chapter 5 Aggregate Columns.

The types of columns that may be used for CSEs and some of their important characteristics are listed in Table 6-1. See *GeoTechTools* for current cost information on the different columns.

Table 6-1. Possible Column Types

Column Type	Range of Allowable Capacity (kips)	Typical Lengths (feet)	Typical Column Diameters (inches)
Timber pile	25 to 100	20 to 60	12 to 18
Steel H pile	100 to 450	30 to 100	10 to 14
Steel pipe pile	175 to 550	30 to 120	10 to 48
Pre-cast concrete piles	100 to 450	30 to 50	10 to 24
Cast-in-place concrete shell (mandrel driven)	100 to 300	20 to 120	10 to 18
Shells driven without mandrel	110 to 300	20 to 75	12 to 36
Continuous Flight Auger piles	75 to 150	20 to 75	12 to 24
Deep mix method (DMM)	90 to 275	20 to 90	24 to 78
Aggregate Columns	25 to 150	10 to 30	24 to 48
VCC	50 to 300	20 to 90	18 to 24
CSV (combined soil stabilization)	5 to 10	10 to 30	5 to 7
CMC	50 to 150	20 to 90	12 to 24

3.1.2 Construction

Column installation typically involves specialized construction equipment. The chapters on Aggregate Columns and Soil Mixing provide information on the construction techniques and equipment requirements for aggregate columns, cement based columns, and soil mix columns. The construction and equipment requirements for CFA piles may be found in GEC 8 (2007). The construction and equipment requirements for driven piles may be found in GEC 12 (2016).

The equipment for most column installation is relatively large and may be heavy. On soft soil projects a working platform may be required to provide access for the equipment. The working platform may include a layer of geosynthetic reinforcement to stabilize the subgrade. This layer of reinforcement is solely for the working platform and should not be included in the LTP analysis. See Chapter 9 Pavement Support Stabilization Technologies for stabilization design and construction guidance.

3.1.3 Column Caps

Column caps are used to decrease the clear span between columns. A CSE with a geosynthetic LTP and column caps is shown in Figure 6-5.

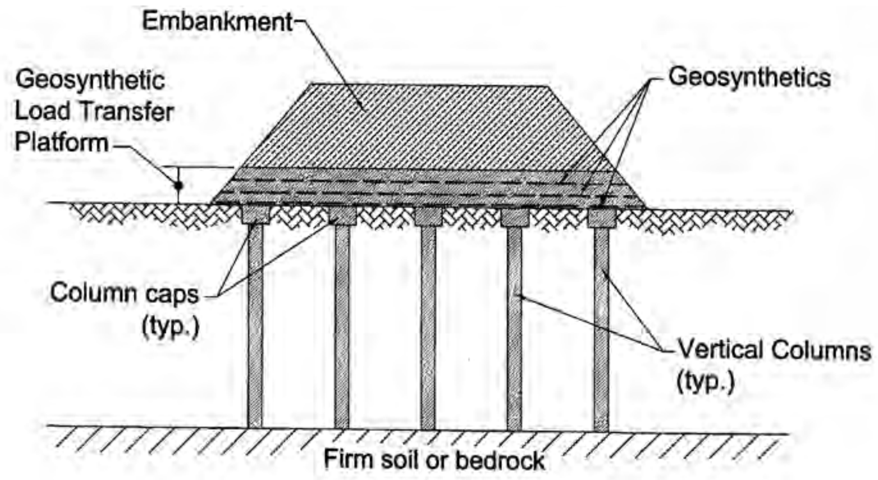


Figure 6-5. CSE with column caps.

The caps usually consist of either cast-in-place or precast concrete. Reinforcing steel may be required. Currently there is little information on the design of column caps. Design issues for column caps are focused on the connection between column and cap, with respect to lateral loads and bending moments (i.e., how are lateral loads determined, where are they applied).

3.2 Load Transfer Platforms

The LTPs covered in this manual consists of select granular structural fill either non-reinforced or reinforced with one or more layers of geosynthetic reinforcement or in situ unreinforced cohesionless soil.

3.2.1 Materials

3.2.1.1 Granular Material

If there is a layer of soil just below the ground surface that is stiff enough and has adequate depth, this layer may act as the LTP. Characteristics of the soil layer and its ability to act as an LTP will be covered in detail in Section 4. If in situ soil at the surface does not have sufficient properties to act as the LTP then backfill material will be necessary to create the LTP. Arching in the LTP soil above the columns is considered an integral component in the transfer of stress from the embankment to the columns. It is, therefore, important that the soils in the zone where the arch is formed be frictional material with high shear strength. Well graded granular fill is considered an ideal material for constructing the LTP. Above the platform, a non-select fill may be used to construct the remainder of the embankment.

3.2.1.2 Geosynthetic Reinforcement

The geosynthetic reinforcement material used to create the load transfer platform has typically been either a single layer of high strength geotextile or geogrid, or several layers of lower strength biaxial geogrid. The type and strength of the geosynthetic reinforcement is a function of the design model used for analysis of the LTP (i.e., catenary or beam), spacing between columns, and height of embankment. Many designers require that a cushion layer of fill be placed between the top of the columns and the geosynthetic reinforcement or a non-woven needle punched geotextile be placed between the top of the pile and the geogrid. The primary function of this cushion is to eliminate abrasion and reduce stress concentrations that would otherwise occur between the top of the column and the reinforcement. Additionally, pile caps should have rounded, and not sharp, edges.

3.2.2 Construction

The geosynthetic reinforcement should be rolled out in the direction indicated on the construction drawings (Figure 6-6).

Figure 6-6. Load transfer platform reinforcement placement.

All wrinkles and slack should be removed prior to fill placement. During fill placement, no construction equipment should be allowed to travel directly on the reinforcement. A minimum of 6 inches of fill should be placed between the reinforcement and any construction equipment.

The requirements for seams shall be considered in the design and the selection of the geosynthetic reinforcement. LTPs constructed to date have used both sewn seams and overlap seams; however, the type of seam should be considered in the design of the LTP.

The select fill (Figure 6-7) used for the LTP should meet project requirements (see Section 5 specifications).

Courtesy James G. Collin

Figure 6-7. Load transfer platform select fill placement.

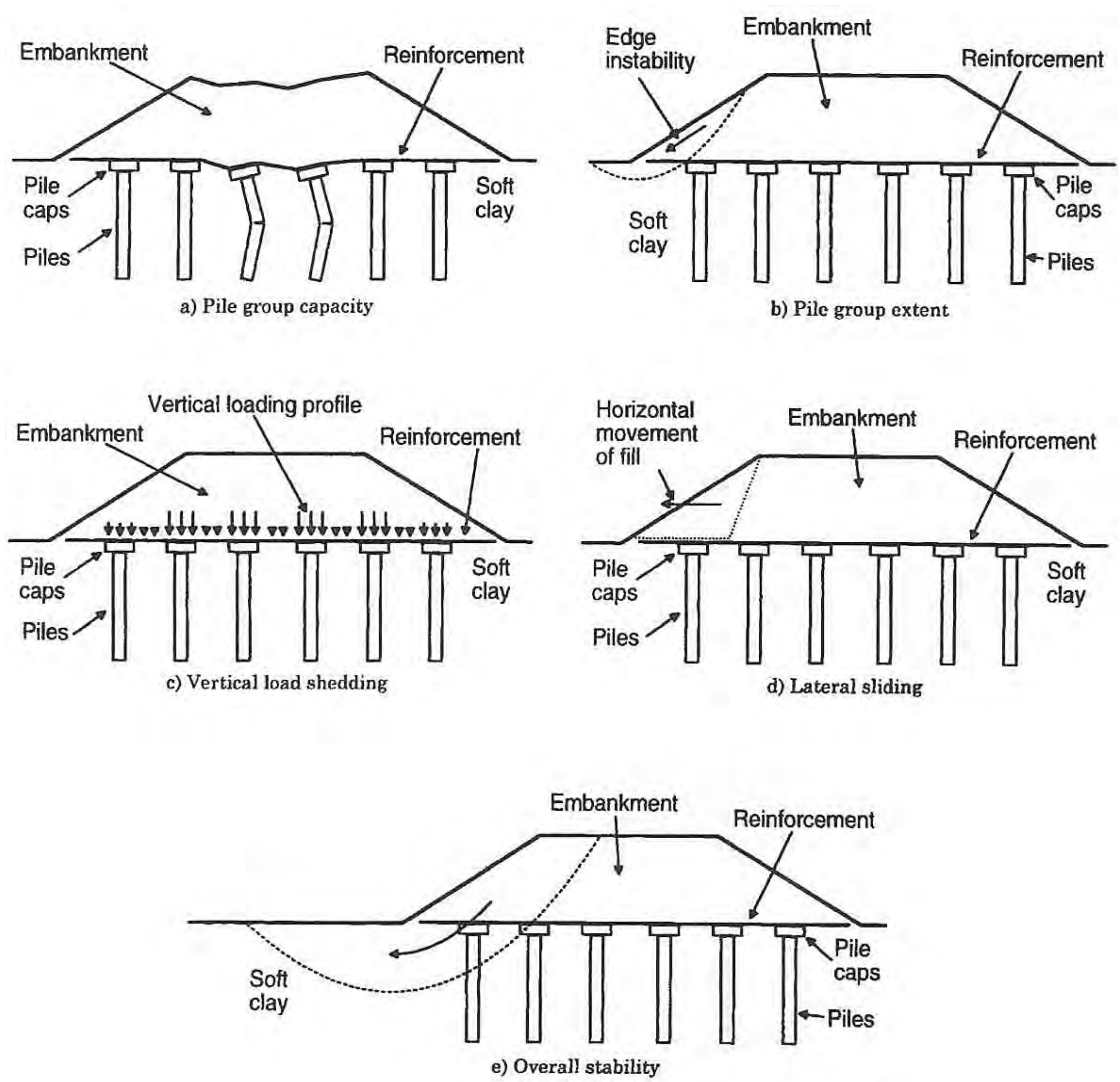
Compaction requirements should be developed considering existing ground conditions. For example if soft soils exist at subgrade it will be difficult to achieve 95% compaction for the first lift of select fill. However, subsequent lifts should be able to achieve the required compaction. The location of the first reinforcement layer should take this into consideration. A layer of reinforcement at subgrade will facilitate achieving project compaction requirements in the first lift when the subgrade soils are soft.

4.0 DESIGN CONCEPTS

The design of CSEs has advanced significantly in the last decade. The early design method predominately used in the United States was the beam method. This method is a semi-empirical method that was developed based on laboratory testing and observation of performance of full-scale structures. This design method was instrumental in the advancement of this technology as it was a relatively easy and straight-forward design method. However, as with most emerging technologies, more sophisticated design methods have been developed. Today, the beam design approach is being replaced by the use of a more analytically correct method that is based on load and displacement compatibility, which we will refer to as the load and displacement compatibility (LDC) method (Sloan et al. 2013). For preliminary designs the beam method is still being used to determine the LTP thickness and reinforcement requirements. However, for a final design, when it has been determined that a geosynthetic reinforced LTP is required, the selection of the reinforcement properties may be based on the preliminary beam design and the settlement analysis performed using the LDC method. The LDC method will be discussed in detail herein. The beam method is presented in Section 4.7. Some specialty contractors and geotechnical consulting firms are developing CSE designs by performing deformation analyses using 2D and 3D numerical modeling. There are numerous finite element and finite difference software programs that are currently available to perform this type of analysis; however, great care and experience is needed to develop a reliable numerical model, select appropriate input parameters, and perform essential quality control checks on the analyses.

4.1 Design Steps

The design of CSEs must consider both strength limit states, and serviceability state failure criteria. The limit state failure modes are shown in Figure 6-8.

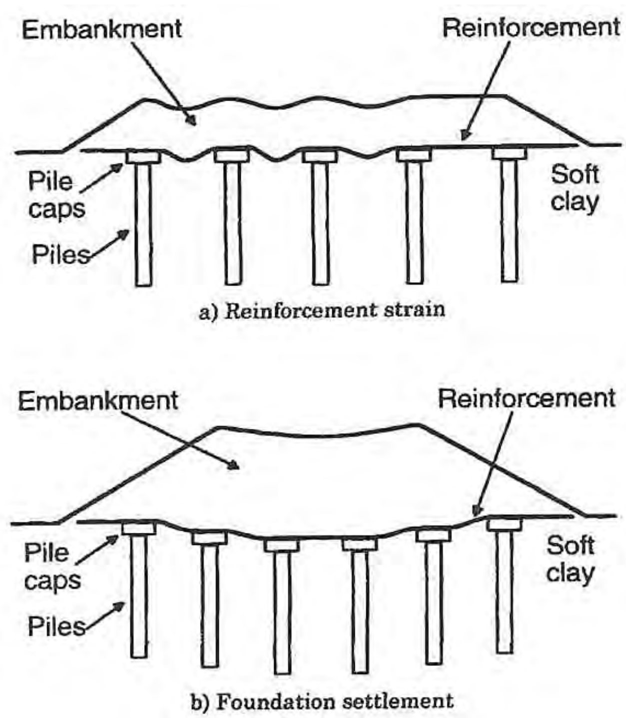


British Standards Institution (BS8006) 2010

Figure 6-8. Limit state failure modes.

The columns must be designed to carry the vertical load from the embankment without failing (Figure 6-8, top row left). The columns are typically assumed to carry the full load from the embankment. The lateral extent of the columns under the embankment must be determined (Figure 6-8, top row right) to prevent slides at the toe of the embankment beyond the outermost column. The foundation soil and/or the load transfer platform must be designed to transfer the vertical load from the embankment to the columns (Figure 6-8, middle row left). The potential for lateral sliding of the embankment on top of the columns must be addressed (Figure 6-8, middle row right). Finally, global stability of the system must be evaluated (Figure 6-8, bottom row).

In addition to strength limit state analyses, serviceability state design must be considered. The strain in the geosynthetic reinforcement used to create the load transfer platform should be kept below some maximum threshold (i.e., typically 5 to 6%) to preclude unacceptable deformation reflection (i.e., differential settlement) at the top of the embankment. Settlement of the columns must also be analyzed to assure that unacceptable settlement of the overall system does not occur, as shown in Figure 6-9.



British Standards Institution (BS8006) 2010

Figure 6-9. Serviceability state.

The general design steps for a CSE are provided below:

1. Estimate preliminary column spacing (see Section 2.3 Feasibility Evaluation).
2. Determine required column load.
3. Select preliminary column type based on column load and site geotechnical requirements.
4. Determine capacity of column to satisfy limit and serviceability state design requirements.
5. Determine extent of columns required across the embankment width.
6. Check critical embankment height criteria and adjust column spacing if required.
7. Determine if LTP is required.

8. If LTP is reinforced, determine reinforcement requirements based on estimated column spacing (steps 1 & 6). Revise column spacing as required.
9. Determine reinforcement requirements for lateral spreading.
10. Determine overall reinforcement requirements based on LTP and lateral spreading.
11. Check global stability.
12. Prepare construction drawings and specifications.
13. Observe construction.

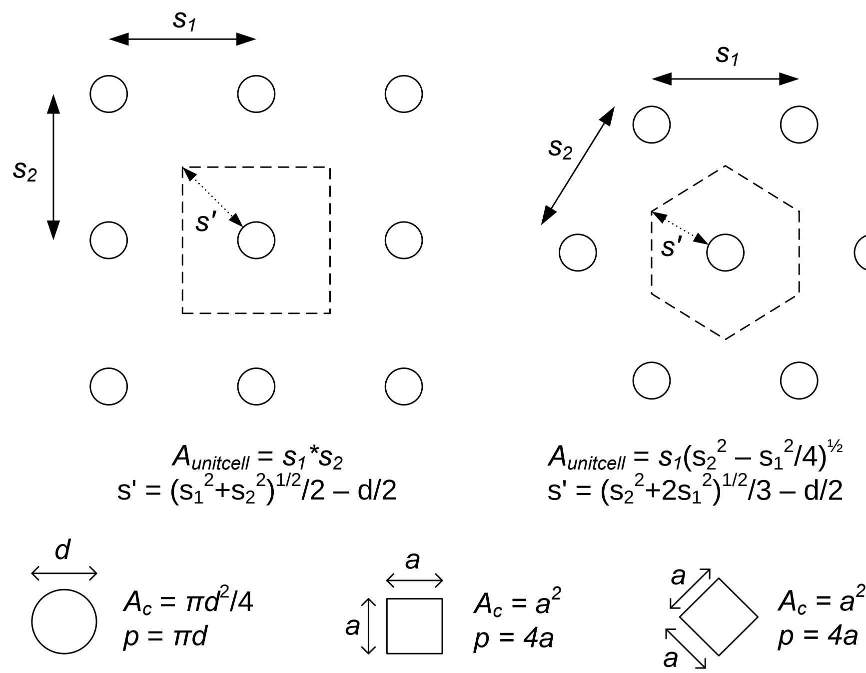
4.1.1 Design CSE with or without LTP

As previously discussed, the design of CSEs has changed significantly over the last decade. In the previous version of this manual, CSEs were designed almost exclusively with geosynthetics reinforced LTPs. The trend now is for CSEs to be designed either with unreinforced LTPs or without an LTP all together. The design methodology presented in this chapter is valid for all three conditions (i.e., no LTP, unreinforced LTP, and reinforced LTP). The selection of the appropriate solution will be based on both serviceability and economics (i.e., does the solution meet the settlement criteria for the project, and what is more economical, a solution without an LTP or one that includes an LTP and therefore is able to use less columns by increasing the column spacing). Note that a LTP, geosynthetic reinforced or unreinforced, is very small cost component of the overall system.

4.2 Column Design

The selection of column type is most often based on constructability, load capacity versus stiffness, and cost. Constructability is discussed in Section 3, and cost will be covered in Section 6. The load that a column is required to carry is typically based on the tributary area for each column. The embankment and any surcharge load are typically assumed to be carried in their entirety by the columns.

For the purposes of determining the design vertical load in the column, it is convenient to associate the tributary area of soil surrounding each column, as illustrated in Figure 6-10.



To convert between square and round columns, an equal area conversion is recommended where a=0.866d

Sloan et al. 2013

Figure 6-10. Column layout and definition sketch for inputs to critical height and adapted Terzaghi method.

Although the tributary area forms a polygon about the column, it can be closely approximated as an equivalent circle having the same total area. For a square column pattern, the effective diameter (diameter D_e) is equal to 1.13 times the center-to-center column spacing. For a triangular column pattern, the effective diameter is equal to 1.05 times the center-to-center column spacing (typical center-to-center column spacing ranges from 5 to 10 feet).

The required design vertical load (Q_r) in the column is determined according to the following equation:

$$Q_r = \pi \left({}^{D_e}\!/_{2} \right)^2 (\gamma H + q)$$

[Eq. 6-1]

where,

D_e	=	effective tributary area diameter of column
H	=	height of embankment
q	=	live and dead load surcharge (typically 250 psf)

γ = unit weight of the embankment soil

This Q_r is the unfactored or nominal load. The range of required column loads for a 5 feet center-to-center column spacing ranges from approximately 25 to 75 kips for embankment heights ranging from 10 to 30 feet. The required load, with a 10 feet center-to-center column spacing, is approximately 90 to 270 kips for embankment heights of 10 to 30 feet. After determining the required load, Q_r, in the column, Table 6-1 (presented in Section 3) together with site subsurface conditions, project-specific constraints, and cost considerations, may be used to select a column type that will provide the required capacity.

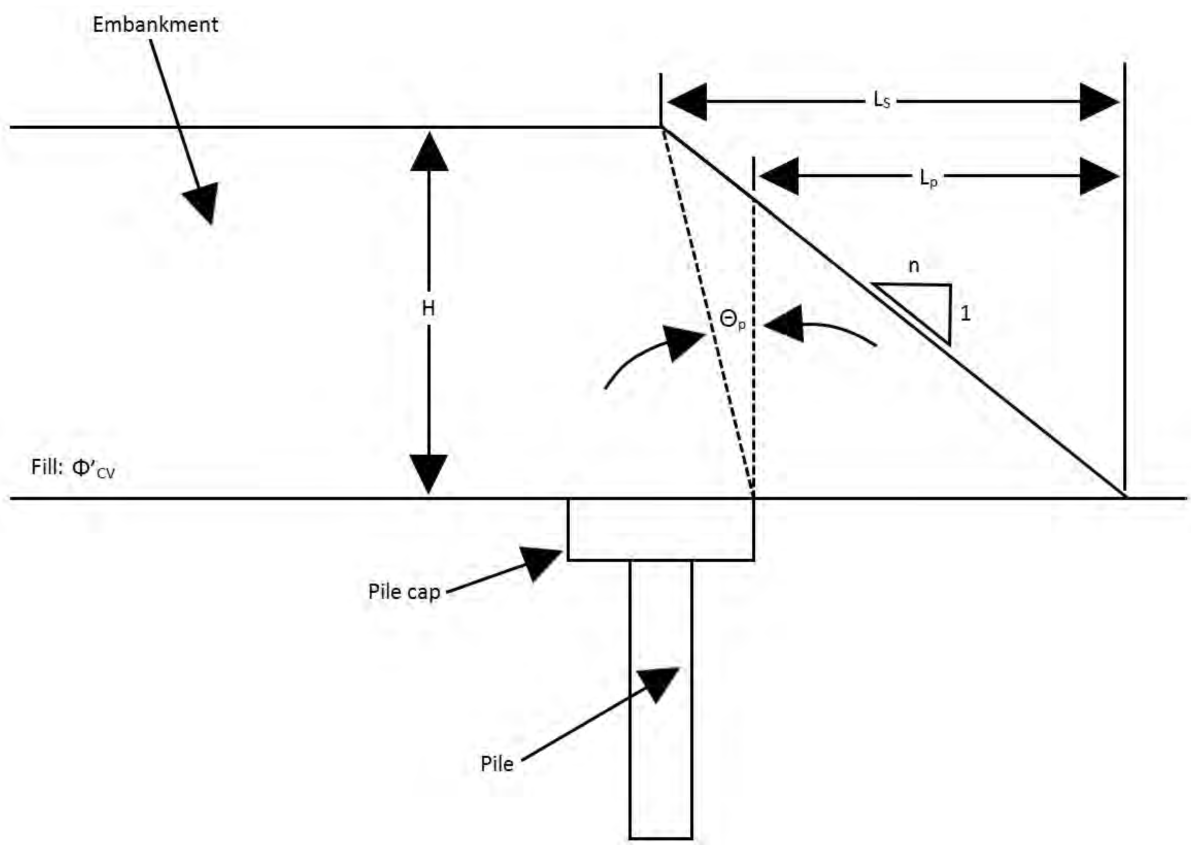
The design of concrete, steel, and timber piling is well established. Design guidelines have been developed by FHWA for driven piles and may be found in GEC 12 (2016). For the design of timber piles, the reader is also referred to *Timber Pile Design and Construction Manual,* Timber Piling Council (Collin 2002). The design of continuous flight auger piles may be found in GEC 8 (2007).

Soil mix columns and aggregate columns are covered in the other chapters of this manual. The vertical load capacity design of cement based columns is typically performed by the contractor. The design verification for these systems is typically achieved with a static load test. A listing of potential columns for this application, and typical design loads and lengths for each, are listed in Table 6-1.

4.3 Lateral Extent of Columns

The lateral extent of the column system across the width of the embankment should extend sufficiently close to the edge of the embankment to ensure that any instability or differential settlement that occurs outside the column supported area will not be problematic (Figure 6-8b). There are several approaches that may be used to check the edge stability. The computer software developed for FHWA for the design of both reinforced and unreinforced slopes and embankments, ReSSA, is an excellent tool for checking edge stability.

The British Standards Institution Code of Practice (herein referred to as BS8006 [2010]) requires that the columns extend to within a minimum distance (L_p) of the toe of the embankment to prevent settlement of the unsupported edge of the embankment from affecting the crest of the embankment. The terms for determining the lateral extent of the columns are shown in Figure 6-11.



BS8006 2010

Figure 6-11. Lateral extent of columns.

L_p is determined from the following equation:

$$L_p = H\left(n - \tan\theta_p\right)$$ [Eq. 6-2]

where,

n	=	side slope of the embankment
θ_p	=	is the angle (from vertical) between the outer edge of the outer-most column and the crest of the embankment [$\theta_p = (45-\phi_{emb}/2)$]
φ_{emb}	=	effective friction angle of embankment fill

4.4 Lateral Spreading

The potential for lateral spreading of the embankment must be analyzed (Figure 6-12).

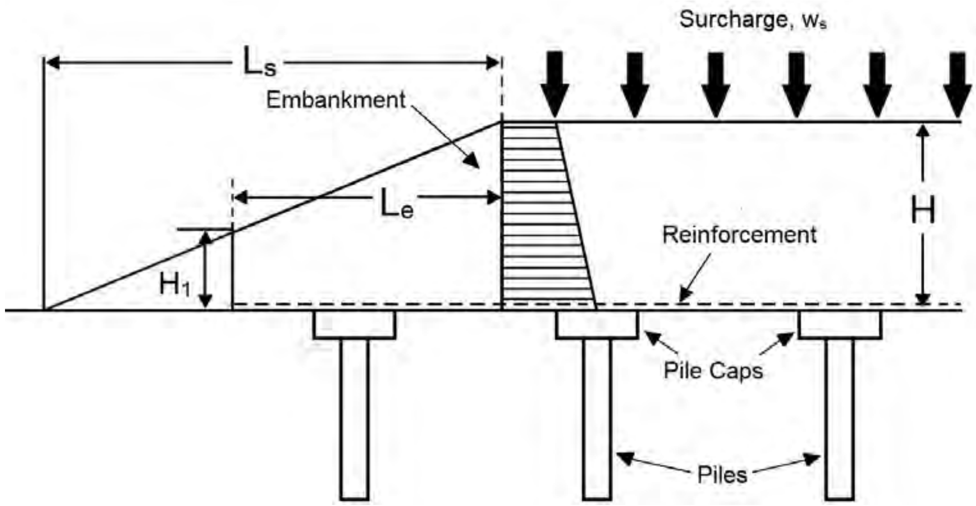


Figure 6-12. Lateral spreading.

The resistance to lateral spreading is provided by the shear strength of the foundation soils at the bottom of the embankment. If inadequate shear resistance is provided by the foundation soils then a geosynthetic reinforcement may be added to provide the required resistance without resorting to battered piles beneath the embankment slope. This is a critical aspect of the design, as many of the vertical columns that are appropriate for column supported embankments are not capable of providing adequate lateral resistance to prevent spreading of the embankment without failing.

The subgrade soil must be adequate or geosynthetic reinforcement must be designed to resist the horizontal force due to the lateral spreading of the embankment. The required tensile force to prevent lateral spreading (P_{Lat}) of the embankment is determined from the following equation:

$$P_{Lat} = K_a \left[\gamma \left(H^2 / 2 \right) + q\,H \right] \quad \text{[Eq. 6-3]}$$

where,

K_a = coefficient of active earth pressure = $\tan^2 (45^o - \phi_{emb}/2)$

The resistance to lateral spreading without a geosynthetics reinforcement is determined by Equation 6-4.

$$R_{ls} = (L_s) S_u \quad \text{[Eq. 6-4]}$$

where,

S_u	=	undrained shear strength of the foundation soil
L_s	=	length of the side slope of the embankment (see Figure 6-11)

A factor of safety for lateral spreading (R_{ls}/P_{Lat}) of 1.5 is recommended. If an adequate factor of safety cannot be achieved, a geosynthetic reinforcement layer should be added. The reinforcement is typically designed to resist the entire lateral spreading force (P_{Lat}). The reinforcement long-term design strength (T_{ls}) should be greater than P_{Lat}. (See GEC 11 (2009) for quantifying T_{ls}.) Multiple layers of reinforcement may be used to resist the lateral spreading force.

$$T_{ls} \geq P_{Lat} \qquad \text{[Eq. 6-5]}$$

The minimum length of reinforcement (L_e) beyond the crest of the embankment towards the toe necessary to develop the required strength of the reinforcement without the side slope of the embankment sliding across the reinforcement is determined using the following equation:

$$L_e = \frac{P_{Lat}}{0.5\gamma H c_{iemb} \tan\phi_{emb}} \qquad \text{[Eq. 6-6]}$$

where,

c_{iemb}	=	coefficient of interaction for sliding between the geosynthetic reinforcement and embankment fill

4.5 Critical Height

Avoiding differential settlement at the surface of a CSE is often important, for example, to provide good ride quality and to prevent distress to overlying structures. Factors that influence differential surface settlements include column spacing, column diameter, embankment height, quality of subgrade support relative to column stiffness, and loading acting on the embankment surface. For example, differential surface settlement is likely for a relatively low embankment with wide column spacing and poor subgrade support. Differential surface settlement is unlikely for a high embankment with close column spacing and good subgrade support. In this chapter, the term *critical height* is defined as the embankment height above which differential settlements at the base of the CSE do not produce measurable differential settlement at the embankment surface. This definition is similar to Naughton's (2007) use of critical height to refer to the vertical distance from the top of the pile caps to the plane of equal settlement in the embankment. Other authors use critical height in other ways, e.g., Horgan and Sarsby (2002) and Chen et al. (2008) use

critical height to refer to the height above which all additional loads due to fill and surcharge are distributed completely to the pile caps.

For CSEs without subgrade support, McGuire (2011) found that the critical embankment height, H_{crit}, depends on the column diameter and spacing, and it is not significantly affected by the relative density of the embankment fill or the use of geosynthetic reinforcement in the load transfer platform, with $H_{crit} = 1.15s'+1.44d$, where s' is defined in Figure 6-10. The critical height from Sloan's (2011) field-scale tests is in good agreement with McGuire's (2011) findings and it is also in good agreement with the more conventional relationship of $H_{crit} = 1.5(s-a)$ for square column arrays because Sloan's tests were performed near where the two expressions for H_{crit} intersect. The approach recommended in the *GeoTechTools* CSE design document is to use the larger value of H_{crit} estimated by these two relationships, as provided below in Equation 6-7.

$$H > H_{crit} = \max\begin{Bmatrix} 1.5(s-a) \\ 1.15s'+1.44d \end{Bmatrix} \quad \text{[Eq. 6-7]}$$

In cases where a square array of square pile caps is used and the embankment height is fixed by the difference between the embankment subgrade elevation and roadway elevation, the minimum center-to-center column spacing can be estimated by Equation 6-8. If the pile caps are round, 0.886d can be substituted for the pile cap width, a, in Equation 6-8.

$$s \le 1.2(H-a) \quad \text{[Eq. 6-8]}$$

Equation 6-9 and 6-10 are for an isosceles and an equilateral triangular array, respectively.

$$s \le 1.4(H-a) \quad \text{[Eq. 6-9]}$$

$$s \le 1.5(H-a) \quad \text{[Eq. 6-10]}$$

4.6 Load Transfer Platform Design

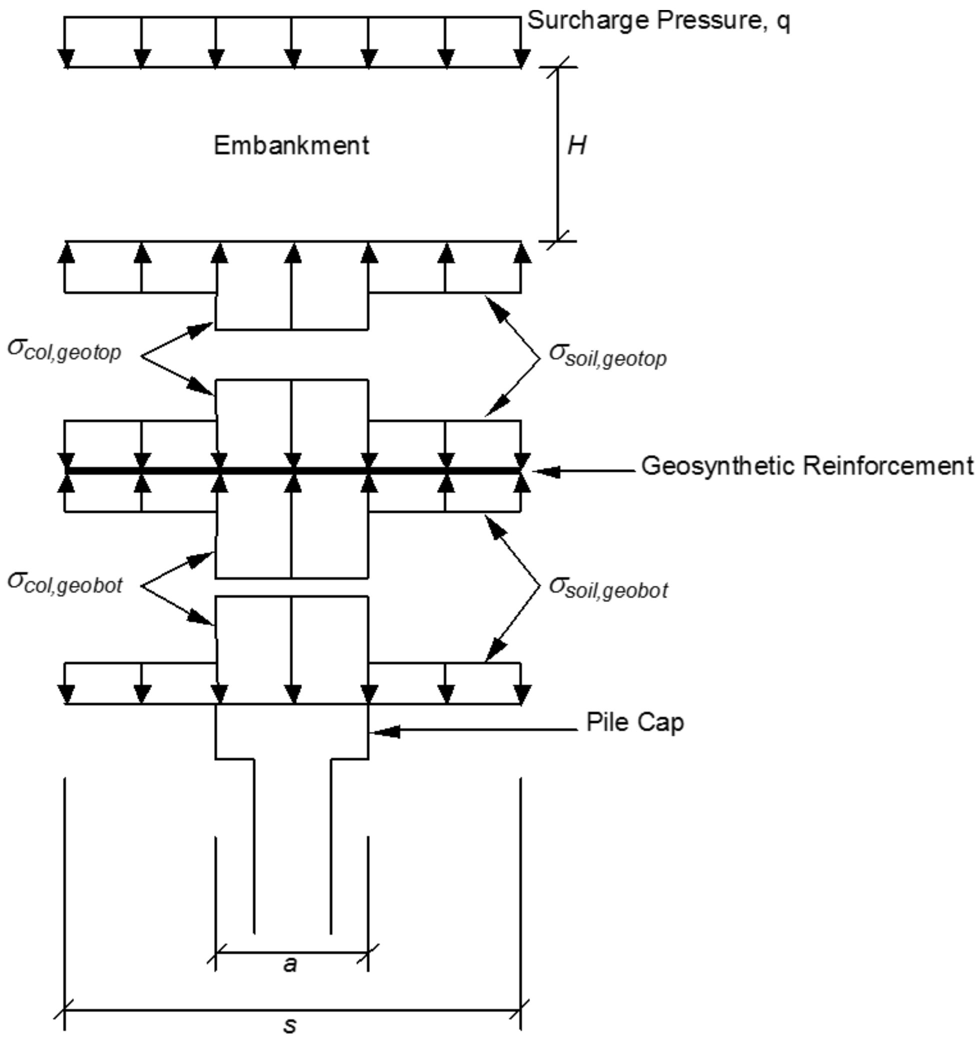
In order for the CSE design to be effective, the embankment load must be transferred to the columns without excessive deformations occurring at the surface of the embankment. There are over a dozen design methods currently available to design the LTP for CSEs. A practical method that models the actual load transfer mechanisms is the load displacement compatibility (LDC) method. This method is the focus within this section.

Smith (2005) and Filz and Smith (2006, 2007) developed a load-displacement compatibility method for analyzing the net vertical load that acts on the geosynthetic reinforcement in the LTP. Although the method was originally developed for geosynthetic reinforced LTPs it can

be used to analyze un-reinforced LTPs as well. Essential features of the LDC method include:

- Vertical load equilibrium and displacement compatibility are assumed at the level of the geosynthetic reinforcement to calculate the load distribution among the columns, the soft soil between columns, the geosynthetic, and the base of the embankment above columns and between columns.
- An axisymmetric approximation of a unit cell is employed for calculating the vertical load acting on the geosynthetic reinforcement, as also employed by Han and Gabr (2002) and others.
- A 3D representation of the geosynthetics-reinforced CSE system and a parabolic deformation pattern of the geosynthetic between adjacent columns is assumed for the purpose of calculating the tension in the geosynthetic, as also employed by BS8006 (2010) and others.
- The LDC method was developed for round columns or square pile caps in a square array.
- Nonlinear response of the embankment is incorporated by providing linear response up to a limit state, at which point additional differential base settlement produces no further load concentration on the columns. The limit state is determined using the Adapted Terzaghi Method described below.
- Linear stress-strain response of the geosynthetic is assumed, but because large displacements of the geosynthetic are involved, the load-displacement relationship for the geosynthetic deformation is nonlinear. Iterations can be performed to approximate nonlinear response of the geosynthetic material.
- Nonlinear compressibility of clay soil between columns is represented using the compression ratio, recompression ratio, and preconsolidation pressure.
- Slippage is allowed between the soil and the column when the interface shear strength is exceeded.

An exploded profile view of a unit cell, including the vertical stresses at the contacts above and below the geosynthetic reinforcement is shown in Figure 6-13.



Filz and Smith 2006

Figure 6-13. Definition sketch for load and displacement compatibility (LDC) method.

Vertical equilibrium of the system shown in Figure 6-13 is satisfied when:

$$\gamma H + q = a_s \sigma_{col,geotop} + (1 - a_s)\sigma_{soil,geotop} = a_s \sigma_{col,geobot} + (1 + a_s)\sigma_{soil,geobot} \qquad \text{[Eq. 6-11]}$$

where,

γ	=	unit weight of the embankment soil
H	=	height of the embankment
q	=	surcharge pressure
a_s	=	area replacement ratio = $A_c/A_{unitcell}$

$\sigma_{col,geotop}$ = average vertical stress acting down on the top of the geosynthetic in the area underlain by the column

$\sigma_{soil,geotop}$ = average vertical stress acting down on the top of the geosynthetic in the area underlain by the soil foundation

$\sigma_{col,geobot}$ = average vertical stress acting up on the bottom of the geosynthetic in the area underlain by the column

$\sigma_{soil,geobot}$ = average vertical stress acting up on the bottom of the geosynthetic in the area underlain by the soil foundation

Load-deflection relationships were developed for: (i) the embankment settling down around the column or pile cap; (ii) the geosynthetic reinforcement deflecting down under the net vertical load acting on the area underlain by soil; and (iii) the soil settling down between the columns. The relationships are only described in conceptual terms here; however, supporting equations and additional details are presented by Filz and Smith (2006). The composite foundation system consisting of the columns and the soil between the columns is discretized, and the simultaneous nonlinear equations can be solved numerically using a spreadsheet program.

The load-deflection relationship for the embankment settling down around the column or pile cap is assumed to be linear up to the maximum load condition. The linear part is approximated using a linear solution for displacement of a circular loaded area on a semi-infinite mass (Poulos and Davis 1974). As indicated previously, square pile caps of width, a, can be approximated as circular pile caps with diameter, d, such that the piles cap areas are the same ($a = 0.866d$). The limiting stress condition in the embankment above the geosynthetic reinforcement is established using the Adapted Terzaghi Method (Russell and Pierpoint 1997) with a lateral earth pressure coefficient, K, of 0.75, which is between the values of 1.0 used by Russell and Pierpoint (1997) and 0.5 used by Russell et al. (2003). Other realistic methods for determining the limiting condition, such as the Hewlett and Randolph (1988) Method or the Kempfert et al. (2004a, 2004b) Method could also be used to establish the limiting condition for settlement of the embankment down around the columns or pile caps.

The geosynthetic deflects down under the net vertical load applied over the area underlain by soil. The geosynthetic load-deflection relationship was developed based on analyses of a uniformly loaded annulus of linear elastic membrane material with the inner boundary pinned, which represents the support provided by the column, and with the outer boundary free to move vertically but not laterally, which represents the axisymmetric approximation of

lines of symmetry in the actual three-dimensional configuration of a column-supported embankment. The details of the analyses and the results are presented by Smith (2005) and Filz and Smith (2006).

The settlements of the column and the subgrade soil are determined based on the vertical stress applied to the top of the column or pile, $\sigma_{col,geobot}$, and the vertical stress applied to the subgrade soil, $\sigma_{soil,geobot}$. The column compression is calculated based on a constant value of the column modulus. One-dimensional compression of clay soil located between columns is calculated using the compression ratio, re-compression ratio, and preconsolidation pressure of the soil. If an upper layer of sand is located between the columns, the sand compression is calculated using a constant value of modulus for the sand. If voids are anticipated between the LTP and subgrade soil the support from the foundation soil should be ignored.

As the compressible soil settles down with respect to the stiffer column, the soil sheds load to the column through shear stresses at the contact between the soil and the column along the column perimeter. The magnitude of the shear stress is determined using an effective stress analysis and a value of the interface friction angle between the soil and the column. The vertical stress increment in the soil from the embankment, and surcharge loads, decreases with depth due to the load shedding process until the depth at which the column settlement and soil settlement are equal. An important detail is that the settlement profile of the subgrade soil at the level of the top of the columns is likely to be dish-shaped between columns. The difference between the column compression and the average soil compression is the average differential settlement at subgrade level. To account for the dish-shaped settlement profile between columns, the suggestion by Russell et al. (2003) that the maximum differential settlement at subgrade level may be as much as twice the average differential settlement was adopted. The test results by Demerdash (1996), McGuire (2011), and Sloan (2011) indicated that this is a conservative approximation, and refinement of this approximation may be warranted.

The computational method described above is solved by satisfying vertical equilibrium using Equation 6-7 and requiring that the calculated values of the differential settlement at subgrade level must be the same for the base of the embankment, the geosynthetics if utilized, and the underlying foundation soil. If there is reason to believe that the soft soil between columns will settle more than the geosynthetic deforms, e.g., due to groundwater lowering, then the subgrade soil can be assigned a very high compressibility value to essentially eliminate subgrade support of the geosynthetic. The simultaneous nonlinear equations that describe this computational method have been implemented in a spreadsheet GeogridBridge (Filz and Smith 2006) that is available on *GeoTechTools* at the following

link: http://www.geotechtools.org/technology-display/GeogridBridge/. GeogridBridge has the following features:

- Two different types of embankment fill are allowed so that lower quality fill can be used above the bridging layer.
- Analyses without geosynthetic reinforcement can be performed by setting the value of the geosynthetic stiffness, J, equal to zero.
- The column area and properties can vary with depth so that embankments supported on piles with pile caps can be analyzed.
- The subsurface profile can include two upper sand layers and two underlying clay layers. The preconsolidation pressure for the clay can vary linearly within each clay layer.
- The simultaneous nonlinear equations are solved automatically, and the input and output are arranged so that design alternatives can be evaluated easily.

The LDC method was validated by comparison with numerical analyses that were previously validated by comparison with instrumented case histories and pilot-scale experiments performed by others. In addition, the overall method was validated by direct comparison with instrumented case histories described by Cao et al. (2006) and Almeida et al. (2007). The comparisons are presented by Filz and Smith (2007) and McGuire et al. (2009).

4.6.1 Generalized Adapted Terzaghi Method

The Adapted Terzaghi Method for determining the limiting distribution of stresses acting up on the base of the embankment has several advantages, including that it is in reasonable agreement with: (i) results of numerical analyses and field case histories (e.g., Russell and Pierpoint 1997, Filz and Smith 2006), (ii) other rational methods (e.g., Hewlett and Randolph 1988 or Kempfert et al. 2004a, 2004b, as shown by McGuire and Filz 2008), and (iii) field tests by Sloan (2011). In addition, it is relatively simple.

The Adapted Terzaghi Method, as presented by Russell and Pierpoint (1997) and Russell et al. (2003) applies to a square arrangement of square columns and only one type of fill material in the embankment. This section presents a generalized version of the Adapted Terzaghi Method to accommodate the following:

- Any column arrangement and any pile cap cross-section area. Examples are shown in Figure 6-10.
- Up to two layers of embankment fill so that a higher quality fill in a load transfer platform and a lower quality fill overlying the load transfer platform can both be

represented. This includes differences in unit weight, friction angle, and lateral earth pressure coefficient.

- Limitation of the vertical shearing in the embankment to the portion below the critical height, with treatment of the embankment weight above this level as a surcharge.

The first and second items in the list above are described by Filz and Smith (2006) and Sloan et al. (2011). In the generalized formulation, the two layers of embankment fill are characterized by: $H_{1,2}$ = layer thicknesses as shown in Figure 6-2, $\gamma_{1,2}$ = layer unit weights, $K_{1,2}$ = layer lateral earth pressure coefficients, and $\varphi_{1,2}$ = layer friction angles. The embankment may have a surcharge, q. As indicated in Figure 6-10, p = the perimeter of the column or pile cap, $A_{unitcell}$ = the area of the unit cell around a column, and A_c = the area of the column or pile cap. The area within a unit cell underlain by soil is $A_{soil} = A_{unitcell} - A_c$. Several of these inputs can be combined in the parameter $\alpha_{1,2}$ for each layer:

$$\alpha_{1,2} = \frac{p\,K_{1,2}\tan\phi_{1,2}}{A_{soil}} \qquad \text{[Eq. 6-12]}$$

The average stress acting up on the base of the embankment in the area underlain by soil, which is $\sigma_{soil,geotop}$ in Figure 6-13 and which can be expressed as σ_{soil} for a CSE without geosynthetic reinforcement, is given by Equation 6-13 for $H_1 + H_2 \leq H_{crit}$, by Equation 6-14 for $H_1 \leq H_{crit} \leq H_1 + H_2$, and by Equation 6-15 for $H_{crit} \leq H_1$.

$$\sigma_{soil,geotop}\ or\ \sigma_{soil} = \frac{\gamma_1}{\alpha_1}\left(1-e^{-\alpha_1 H_1}\right) + \frac{\gamma_2}{\alpha_2}\left(e^{-\alpha_1 H_1}\right)\left(1-e^{-\alpha_2 H_2}\right) + q\left(e^{-\alpha_1 H_1}\right)\left(e^{-\alpha_2 H_2}\right) \qquad \text{[Eq. 6-13]}$$

$$\begin{aligned}\sigma_{soil,geotop}\ or\ \sigma_{soil} = {} & \frac{\gamma_1}{\alpha_1}\left(1-e^{-\alpha_1 H_1}\right) + \frac{\gamma_2}{\alpha_2}\left(e^{-\alpha_1 H_1}\right)\left(1-e^{-\alpha_2 (H_{crit}-H_1)}\right) + \\ & \left[q + \left(H_1 + H_2 - H_{crit}\right)\gamma_2\right]\left(e^{-\alpha_1 H_1}\right)\left(e^{-\alpha_2 (H_{crit}-H_1)}\right)\end{aligned} \qquad \text{[Eq. 6-14]}$$

$$\sigma_{soil,geotop}\ or\ \sigma_{soil} = \frac{\gamma_1}{\alpha_1}\left(1-e^{-\alpha_1 H_{crit}}\right) + \left[q + \left(H_1 - H_{crit}\right)\gamma_1 + H_2\,\gamma_2\right]\left(e^{-\alpha_1 H_1}\right) \qquad \text{[Eq. 6-15]}$$

4.6.2 Generalized Parabolic Method

There are at least three methods for calculating tension in the geosynthetic reinforcement in a CSE: the parabolic method (BS8006 2010), the tensioned membrane method (Collin 2004, 2007), and the embedded membrane method (Kempfert et al. (2004a, 2004b). The parabolic method shows good agreement with numerical analyses (Filz and Plaut 2009) and with the

field-scale tests by Sloan (2011). The parabolic method presented in BS8006 (2010) applies to square pile caps in a square array, and it does not incorporate stress-strain compatibility.

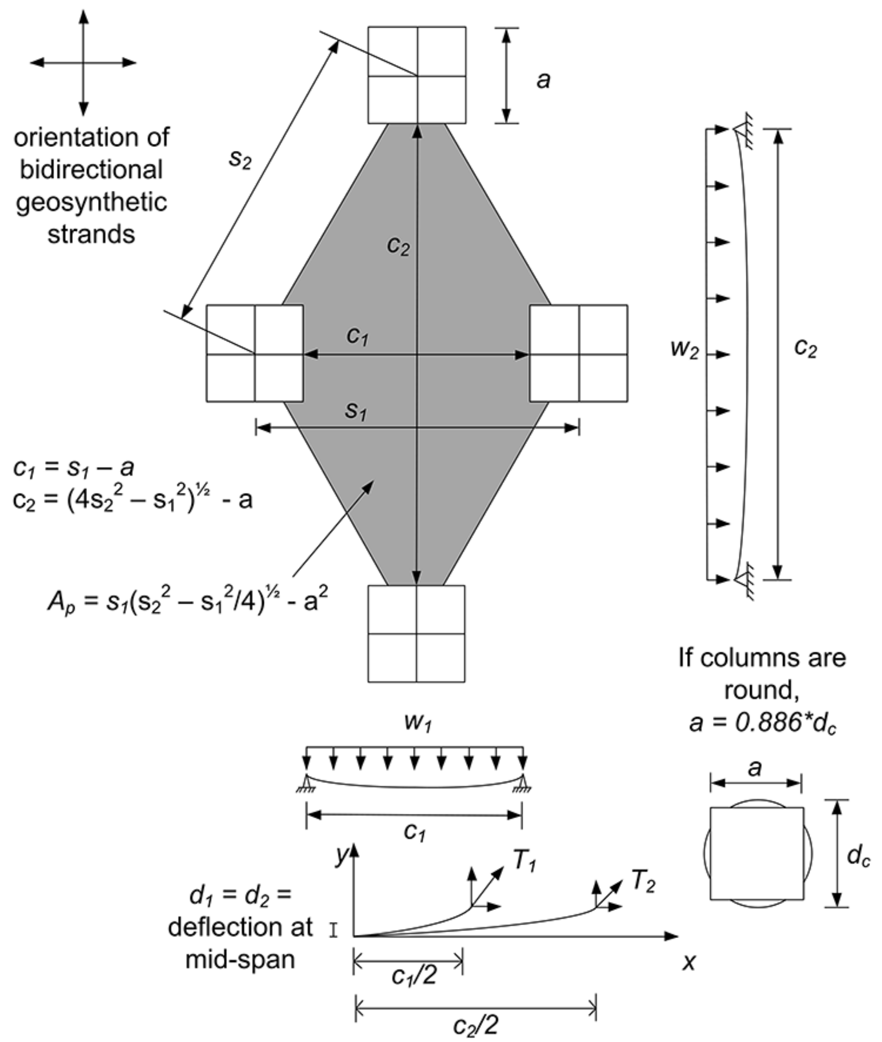
Filz and Smith (2006) presented a solution of the parabolic method with stress-strain compatibility, and Sloan (2011) adapted the method to the geometries shown in Figure 6-10. The solution for biaxial geogrids placed in alignment with a rectangular array of columns is:

$$6T^3 - 6T\left(\frac{\sigma_{net} A_{soil}}{p}\right)^2 - J\left(\frac{\sigma_{net} A_{soil}}{p}\right)^2 = 0 \qquad \text{[Eq. 6-16]}$$

where T = the tension in the geogrid, $\sigma_{net} = \sigma_{soil,geotop} - \sigma_{soil,geobot}$ = the net vertical stress acting on the geogrid, A_{soil} is the area of geogrid in a unit cell underlain by soil ($A_{soil} = A_p$ in Figure 6-14) , p = the column or pile cap perimeter, and J = the sum of the stiffnesses of the geogrid layers. Typically, two to four geogrid layers are used, with the direction of each successive geogrid layer rotated by 90 degrees, so use of an average value of J is justified, even if the values of J are slightly different in the two principal directions of a biaxial geogrid. Equation 6-16 can be solved for the tension T, and the strain in the geosynthetic is given by $\varepsilon = T/J$. Equation 6-16 is recommended for rectangular column arrays with $0.5 \le s_1/s_2 \le 2$, including square arrays for $s_1 = s_2$, where s_1 and s_2 are defined in Figure 6-10.

Equation 6-10 also applies for radially isotropic geogrids, which have relatively uniform stiffness, J, in all directions within the plane of the geogrid, over columns in rectangular or triangular arrays with $0.5 \le s_1/s_2 \le 2$.

For the case of biaxial geogrids aligned over a triangular array of columns, the solution is based on the assumptions shown in Figure 6-14.



Sloan et al. 2013

Figure 6-14. Triangular column arrangements with biaxial geogrid.

The solution for this case is:

$$\frac{2aT_1}{\sqrt{1+J_1/6T_1}}+\frac{2aT_2}{\sqrt{1+J_2/6T_2}}-\sigma_{net}A_{soil}=0 \; and \; \frac{T_1c_1^2}{J_1}=\frac{T_2c_2^2}{J_2}$$

[Eq. 6-17]

which can be solved simultaneously for T_1 and T_2, which can then be used to determine the strains according to $\varepsilon_1 = T_1/J_1$ and $\varepsilon_2 = T_2/J_2$.

4.6.3 Geosynthetic Properties

The two values of the geosynthetic reinforcement used in LTP design are the stiffness, J, and the available long-term strength. These values relate to the serviceability state and to the

strength state, respectively. The stiffness, J, of the geosynthetic should be the long-term stiffness; and should not be confused with the geosynthetic load-displacement relationship determined with quick (or short-term) test methods (e.g., ASTM D6637). The stiffness, J, for LTP design should be defined at the specified, or selected, design life and reinforcement design strain value. Thus, the isochronous creep curve at the design life (e.g., 50 years, 75 years, etc.) is used to define the stiffness, J, as the tensile strength corresponding to design strain value.

The available long-term strength is quantified as the quotient of the quick (or short-term) tensile testing (e.g., ASTM D6637) and the product of creep, installation damage, and durability reduction factors. The procedures to quantify the allowable long-term geosynthetic reinforcement strength values are well established; see Chapter 3 in GEC 11 (2009).

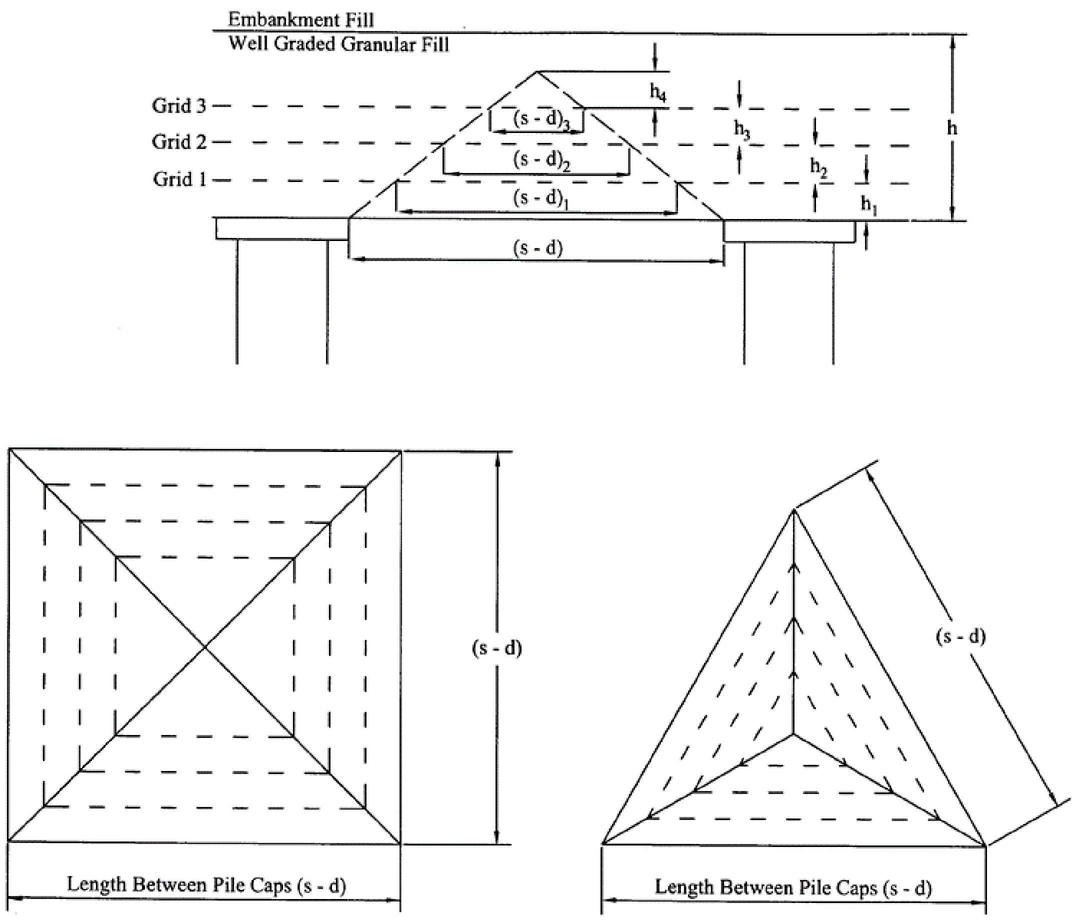
4.7 Beam Method

The beam method is simple and easy to use and is therefore often used for the preliminary design of the LTP when for example the characterization of the soft subgrade soils is not complete enough to provide strength and compressibility characteristics of the soft subgrade soils to use in the LDC method.

The beam method is based on the following assumptions:

- The thickness (h) of the load transfer platform is equal to or greater than one-half the clear span between columns (½ (s-d)).
- A minimum of three layers of extensible (geosynthetic) reinforcement is used to create the load transfer platform.
- Minimum distance between layers of reinforcement is 8 inches.
- Select fill is used in the load transfer platform.
- The primary function of the reinforcement is to provide lateral confinement of the select fill to facilitate soil arching within the height (thickness) of the load transfer platform.
- The secondary function of the reinforcement is to support the wedge of soil below the arch.
- All of the vertical load from the embankment above the load transfer platform is transferred to the columns below the LTP.
- The initial strain in the reinforcement is limited to 5%.

The vertical load carried by each layer of reinforcement is a function of the column spacing pattern (i.e., square or triangular) and the vertical spacing of the reinforcement. If the subgrade soil is strong enough to support the first lift of fill, the first layer of reinforcement is located 6 to 10 inches above subgrade. Each layer of reinforcement is designed to carry the load from the LTP fill that is within the soil wedge below the arch. The fill load attributed to each layer of reinforcement is the material located between that layer of reinforcement and the next layer above (Figure 6-15).



Young et al. 2003

Figure 6-14. Beam method.

The uniform vertical load on any layer (n) of reinforcement (W_{Tn}) may be determined from the following equation:

W_{Tn} = (area at reinforcement layer n + area at reinforcement layer (n+1))/2) (layer thickness) (load transfer platform fill density)/(area at reinforcement layer n)

$$W_{Tn} = \left[(s-d)_n^2 + (s-d)_{n+1}^2\right]\sin 60^\circ \left(\frac{h_n \gamma}{(s-d)_n^2 \sin 60^\circ}\right) \textit{for triangular pattern}$$

[Eq. 6-18]

$$W_{Tn} = \left[(s-d)_n^2 + (s-d)_{n+1}^2\right]\left(\frac{h_n \gamma}{(s-d)_n^2}\right) \textit{for square pattern}$$

[Eq. 6-19]

The tensile load in the reinforcement is determined based on tension membrane theory (Giroud et al. 1990) and is a function of the amount of strain in the reinforcement. The tension in the reinforcement is determined from the following equation:

$$T_{rpn} = W_{Tn}\,\Omega\left(D/2\right)$$

[Eq. 6-20]

where,

D = design spanning for tension membrane

D = $(s\text{-}d)_n$ for square column spacing

D = $(s\text{-}d)_n$ tan 30° for triangular column spacing

Ω = dimensionless factor (see Table 6-2)

Table 6-2. Values of Ω

Ω	Reinforcement Strain (ε)%
2.07	1
1.47	2
1.23	3
1.08	4
0.97	5

Source: Young et al. 2003

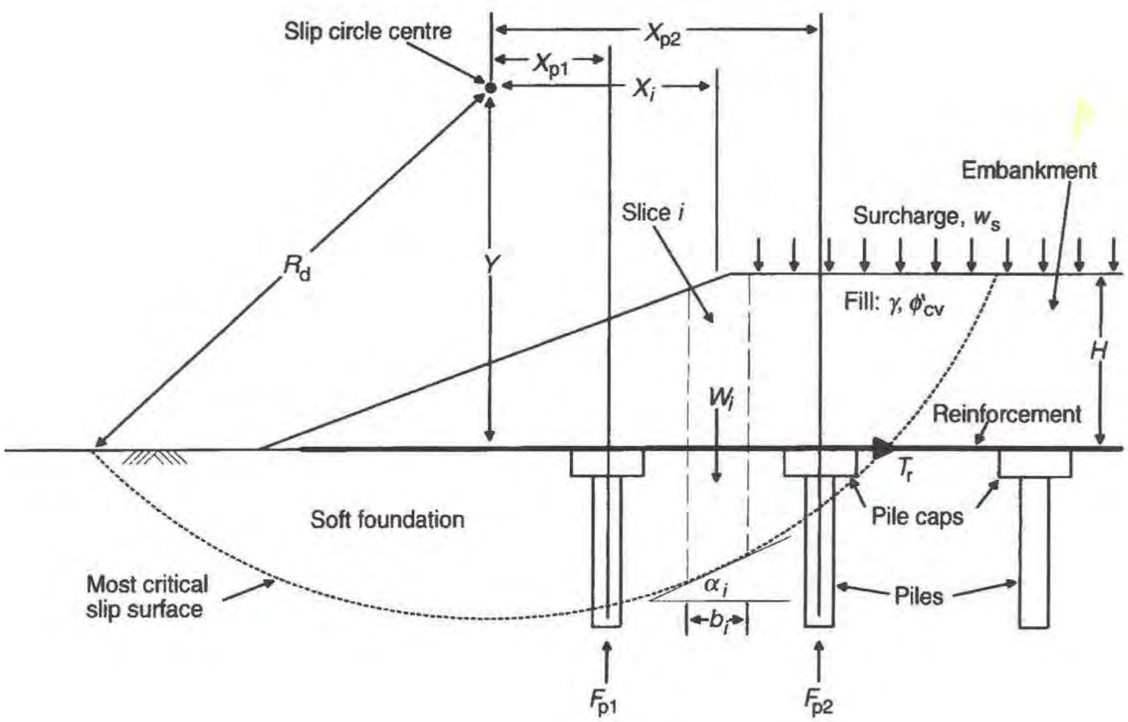
4.8 Reinforcement Total Design Load

Separate geosynthetic reinforcement layers for lateral spreading and for the LTP have been used, or the force requirements for both modes of failure have been combined and a geosynthetic that can resist the sum of the loads has been utilized. The tensile strength for lateral spreading may be relatively high compared to the reinforcement requirements for the LTP. In addition, the lateral spreading load direction is perpendicular to the embankment, requiring geosynthetics with strength in one direction. The load direction for the

reinforcement in the LTP is bi-directional requiring a biaxial geogrid or geotextile. For these reasons, it is generally recommended that separate reinforcements be used to address lateral spreading and for the LTP. The allowable long-term tensile strength of the geosynthetic is used in the lateral spreading computations. As noted under Section 4.6.3, the available long-term strength is quantified as the quotient of the quick (or short-term) tensile testing (e.g., ASTM D6637) and the product of creep, installation damage, and durability reduction factors.

4.9 Global Stability

Global stability of column supported embankments may be evaluated using limit equilibrium computer software, taking into consideration the added shear resistance of the columns and the tensile capacity of the geosynthetic reinforcement. The approach used in the British Standard for incorporating the benefit of the columns and geosynthetic is shown in Figure 6-16.



British Standards Institution (BS8006) 2010

Figure 6-15. Variables used in global stability analysis.

For more guidance on incorporating the benefit of the columns into the global stability analysis see Chapter 5 Aggregate Columns. For guidance on incorporating the benefit of geosynthetic reinforcement in the overall stability of the CSE, see GEC 11 (2009).

When the load from the embankment is effectively transferred to the firm foundation soil below the soft layer, using the procedures outlined above, there is very small potential for a global stability problem and consequentially global stability analyses are not generally required.

5.0 CONSTRUCTION SPECIFICATIONS AND QUALITY ASSURANCE

Like other methods of specialty construction, unless the specifying agency has expertise in the design, construction, and inspection of column supported embankments, it is good practice to specify that the work be accomplished under a performance type specification. If the specifying agency has the necessary experience with the technology, a method specification may be utilized.

5.1 CSE Performance Specification

Performance specifications shall include the design and installation of the columns, as well as the load transfer platforms. Specifications for the various column types are beyond the scope of this chapter. The reader is referred to GEC 12 (2016) for more information on performance specifications for driven piles. For soil mix columns and aggregate columns, see the other chapters of this manual. See *GeoTechTools* for information on cement based columns.

As part of the development of *GeoTechTools*, an extensive evaluation was made of specifications for CSEs. The method and the performance specification presented in the previous version of this document and a draft Minnesota DOT specification for a CSE LTP were reviewed. These specifications, and project experience, were used to develop a guide preferred specification entitled *Column-Supported Embankment Performance Guide Specification* that is intended to be a complete specification containing commentary and instructions that are easily adaptable by the user for a specific project. This guide specification can be accessed in the *GeoTechTools* system (http://www.GeoTechTools.org) under the Technology Information page for Column-Supported Embankments.

The specification shall clearly define the modes of failure that must be analyzed as part of the design/build Contractor's submittal and the required minimum factors of safety. However, the choice of design methods should be left to the Contractor. It is recommended that as part of the approval process the Specifying agency check the contractors design using the LDC method.

5.2 LTP Method Specification

If the specifying agency wants a specific design approach used for the design of the load transfer platform, then a method specification for the LTP, that is complimentary to the CSE performance specification, may be used. Alternatively, a combined performance/method specification for the CSE and LTP could be developed and used. A guide method specification for a LTP can be accessed in the *GeoTechTools* system (http://www.GeoTechTools.org) under the Technology Information page for Column-

Supported Embankments. This specification should be modified as appropriate for the particular requirements of the project.

5.3 Quality Assurance

QC/QA for a column supported embankment project should include verification of the properties and placement of the LTP fill, embankment fill, and the geosynthetic reinforcement. Very large projects may include a budget for an embankment test section. However, as more knowledge of CSEs has been gained over the last decade through case histories, numerical modeling, and the full-scale embankment tests, a need for test embankment sections, even for large projects, has diminished. Some type of settlement and/or lateral displacement monitoring should be included to determine if the embankment performs as expected. Although not covered in this document, industry standard QC/QA procedures for the type of column or pile used for embankment support should be followed.

Pre-production embankment test sections should be considered only on very large projects or where a performance approach specification is used. For large projects, design validation is particularly useful, because a test section may lead to a more economical design. If a performance approach specification is used, then monitoring of the embankment test section will serve as the basis for an acceptable design. Typically the acceptance criteria are based on minimum total and/or differential settlement criteria.

Geosynthetics testing and verification should include:

- Documentation of manufacturer, model number, lot number, and roll number for each roll
- Verify the following properties of the geosynthetic per manufacturer's certified test results: ultimate strength per ASTM D6637 (geogrid) or ASTM D4595 (geotextile), creep resistance per ASTM D5262, durability, and coefficient of interaction for sliding per ASTM D5321 (ASTM 2015).
- Inspection of each role to verify that it is undamaged prior to covering with fill material
- Storage and shipment should be such that the geosynthetic does not receive prolonged exposure to ultraviolet radiation prior to covering
- Observation to verify removal of deleterious materials prior to placement of geosynthetic reinforcement

- Observation of geosynthetic placement to verify it is taut, unless sagging is prescribed by the design method and construction notes to enhance arching in the embankment fill
- Observation to verify that equipment is not operated directly on the geosynthetic and minimum fill thickness is placed before equipment is operated over geosynthetic; equipment should not make sharp turns
- Observation to verify there are no large piles of fill material on top of the LTP which may cause a local bearing capacity failure
- Observation to verify proper orientation, overlap, and elevation within the embankment
- If geotextile seams are specified, the seams should be placed up and every stitch should be inspected.

Verification for the LTP and embankment fill should include:

- Grain size distribution of fill material(s) to verify it meets the specified gradation (frequency of testing determined by state DOT recommendations typical for embankment fill projects)
- Atterberg limits to verify liquid limit and plasticity index are below the specified maximum values (frequency of testing determined by state DOT recommendations typical for embankment fill projects)
- Modified Proctor compaction tests to determine the maximum dry unit weight and the optimum moisture content (for use in calculating relative compaction and determining the allowable range of moisture contents), or minimum and maximum density tests (for use in calculating relative density for granular fill placement)
- In situ density verification with nuclear gage, sand cone, balloon densometer, or other reliable method; the specific method of density testing and frequency should follow guidelines typical of the DOT in the state where the project is located
- Observation to verify maximum lift thickness is not exceeded (recommend 10 inches for large compaction equipment and 6 inches for hand operated equipment).

The following monitoring is recommended:

- Surface survey to confirm the finished embankment elevation; periodic resurvey to quantify total and differential settlement
- Settlement plates at the elevation of the geosynthetic reinforcement should be considered to monitor settlement during construction

- Inclinometers at the embankment toe should be considered to monitor lateral displacement.

6.0 COST DATA

This section presents guidelines for preparing budget estimates in order that the economic feasibility of the CSE may be evaluated. Readers are referred to *GeoTechTools* for additional information and guidance on preparing preliminary cost estimates. This section will provide cost information on the three main components of a CSE, the cost of a construction working platform (if required), the columns and the LTP.

6.1 Access and Mobility

For many of the sites where CSE technology is utilized, the existing surface soils are so weak that a working platform is required in order for equipment to be able to move around the site. The working platform typically consists of a geosynthetic reinforcement and a bridging layer of aggregate. The design of the working platform is beyond the scope of this chapter. However, readers are encouraged to see *GeoTechTools* and Chapter 9 Pavement Support Stabilization Technologies for information on the design of working platforms.

Estimating the cost of the working platform is relatively straightforward. The components are the geosynthetic reinforcement and aggregate. The developers of *GeoTechTools* reviewed DOT bids between 2005 and 2010 and determined the range of cost for geosynthetics used for working platforms varied between $1.00 to $3.50 per square yard, including delivery, overlaps, and waste. The cost for granular fill varied drastically from $7.00 to $20.00 per ton delivered, depending on what part of the country the project is located.

Once the thickness of the working platform is estimated (see *GeoTechTools*), the cost for the working platform can easily be estimated. The equipment and labor costs to construct the platform may be estimated to be about equal to the cost of the platform materials.

6.2 Column Cost

Information in Table 6-1 can be used to perform a preliminary estimate of the cost of the columns. The unit cost is for production column installation. Additional cost that should be considered are mobilization, and column verification load tests. Quality assurance testing and observation should also be considered.

6.3 LTP Cost

The components of the load transfer platform are the geosynthetic reinforcement, the select LTP Fill and the labor to install these materials. The geosynthetic reinforcement cost that has been used on several projects constructed between 2005 and 2010 varies between $8.35 to 12.00/yd^2, including delivery, overlaps, and waste. The cost for granular fill varied

drastically from \$7.00 to \$20.00 per ton delivered, depending on what part of the country the project is located. The thickness of the load transfer platform may be estimated for preliminary cost purposes to be one-half the clear spacing between columns ((s-d)/2). The equipment and labor costs to construct the LTP may be estimated to be about equal to the materials costs.

A preliminary cost estimate for a column spacing of 10 feet, with a 20-inch column diameter, and a unit cost of \$10.00/yd^2 for the reinforcement and \$12.00/ton for the select fill is shown below:

- Reinforcement cost per yd^2 plan area of load transfer platform = \$10/yd^2
- Select Fill Cost per yd^2 plan area
    - Estimated thickness of load transfer platform (s-d)/2 = (10-1.67)/2 = 4.1 ft
    - Estimated weight of select fill/sf plan area = (4.1 ft)(125 pcf) = 520 psf
    - Estimated cost of select fill/sf plan area (520 psf)(\$6/1,000 lbs) = \$3.12/ft^2 ≈ (\$28/yd^2)
- Material costs = \$38 yd^2
- Labor costs = \$38/yd^2

The total estimated cost for load transfer platform = \$ 76/yd^2 of plan area of LTP.

7.0 CASE HISTORIES

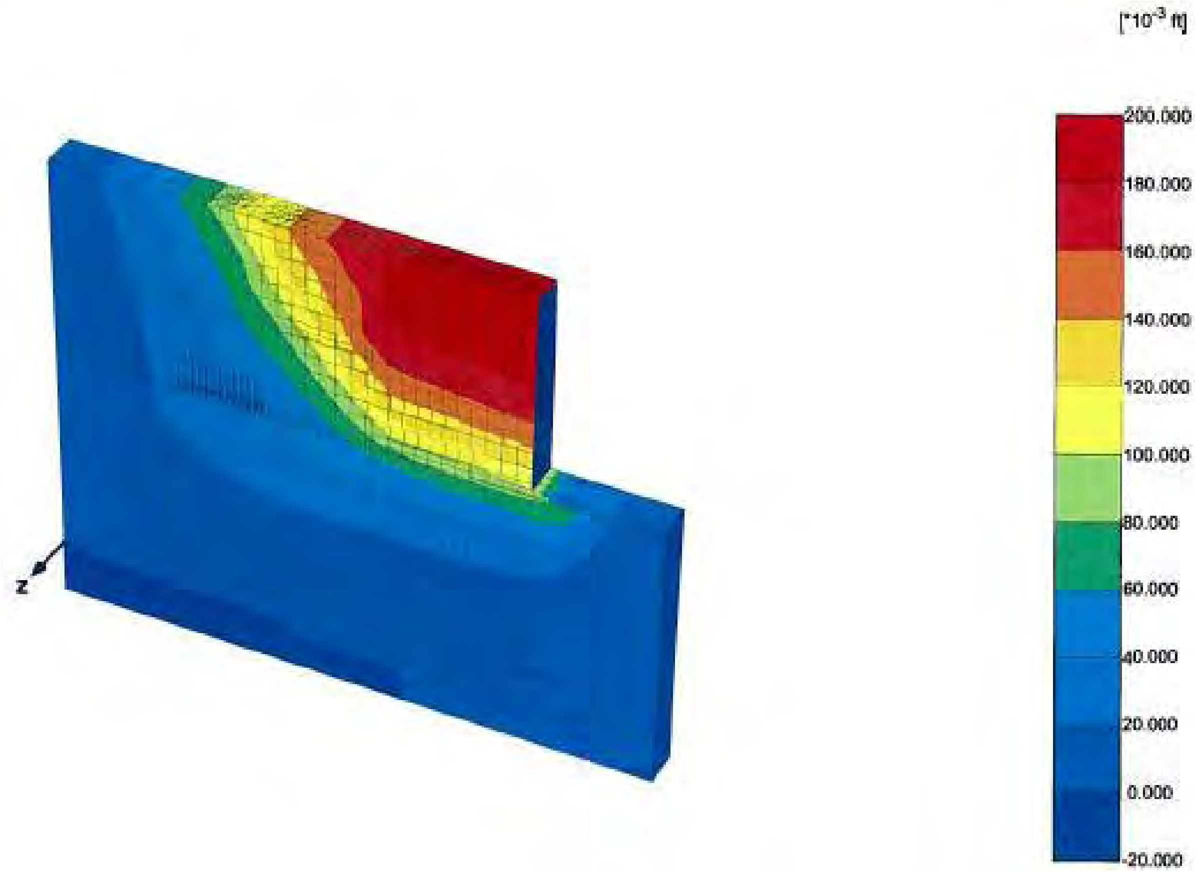
7.1 Garden State Parkway Bridge over Mullica River

The basic information for the Garden State Parkway Bridge over the Mullica River is as follows:

- Project Location: Port Republic, NJ
- Owner: NJ Turnpike Authority
- Engineer(s): Parsons Brinckerhoff, Inc.
- Contractor: Agate Construction Co
- Ground Modification Subcontractor: Menard Group USA
- Year Constructed: 2010

7.1.1 Project Summary

When the New Jersey Turnpike Authority (NJTA) widened the roadway from two to three lanes for the approaches to the Garden State Parkway Bridge over the Mullica River, a two-stage MSE retaining wall was designed to support the embankment and minimize encroachment on wetlands. The plan initially specified stone columns to provide ground improvement to support the MSE walls. The ground modification subcontractor proposed a value engineered alternative using Controlled Modulus Columns (CMCs) instead of stone columns, which would allow the construction of a one-stage MSE wall rather than a two-stage wall. The NJTA, along with the general contractor, selected the CMC alternative. The CMC design was performed using a large scale 3D finite element model, with several supporting 2D finite element models (Figure 6-17). These advanced computational methods helped to optimize the design and meet the target performance requirements for the project.



Courtesy Menard Group USA

Figure 6-17. 3D numerical model of MSE wall and CMCs.

7.1.2 Ground Conditions

The site had a variable soil profile, with varying depths of controlled embankment fill in the upper layers, underlain by organics and sand below. The columns were installed through the organics and were founded in the dense sand at depth. Depths for the columns varied from 25 to 50 feet.

7.1.3 Ground Modification Solution

NJTA used CMCs for ground modification of 1,400 linear feet of MSE wall on the south approach and 2,600 linear feet on the north approach with a total treatment area of approximately 126,000 square feet. The ground modification subcontractor installed a total of 2,129 columns for the Mullica River Bridge Project in two phases (Figure 6-18).

Courtesy Menard Group USA

Figure 6-18. CMC installation Phase 2.

During phase one, columns were installed from a lower elevation in the strip zone of the MSE walls. After the wall was partially constructed the remaining columns were installed from a higher elevation for the embankment support. The MSE retaining walls were designed by wall system supplier. The ground modification was completed on schedule in the summer of 2010.

7.2 US 61 Bridge over Mississippi River

The basic information for the US 61 Bridge over the Mississippi River is as follows:

- Project Location: Hastings, MN
- Owner: Minnesota Department of Transportation
- CSE Engineer: Dan Brown and Associates
- Contractor: Lunda-Ames Joint Venture
- Year Constructed: 2010-2014 (CSE was constructed in 2010 and 2011)

7.2.1 Project Summary

A $130M design-build replacement bridge project was completed in 2014 carrying four lanes of traffic on Highway 61 as well as a pedestrian trail over the Mississippi River in Hastings, MN. A 35-foot tall approach embankment located adjacent to the existing bridge embankment on the north side of the project was located in an area containing multiple deep compressible strata (Figure 6-19).

Strict performance requirements regarding serviceability and global stability combined with the tight construction schedule dictated ground modification beneath the embankment. Accordingly, after evaluating several different alternatives, the design-build team elected to design and construct a CSE.

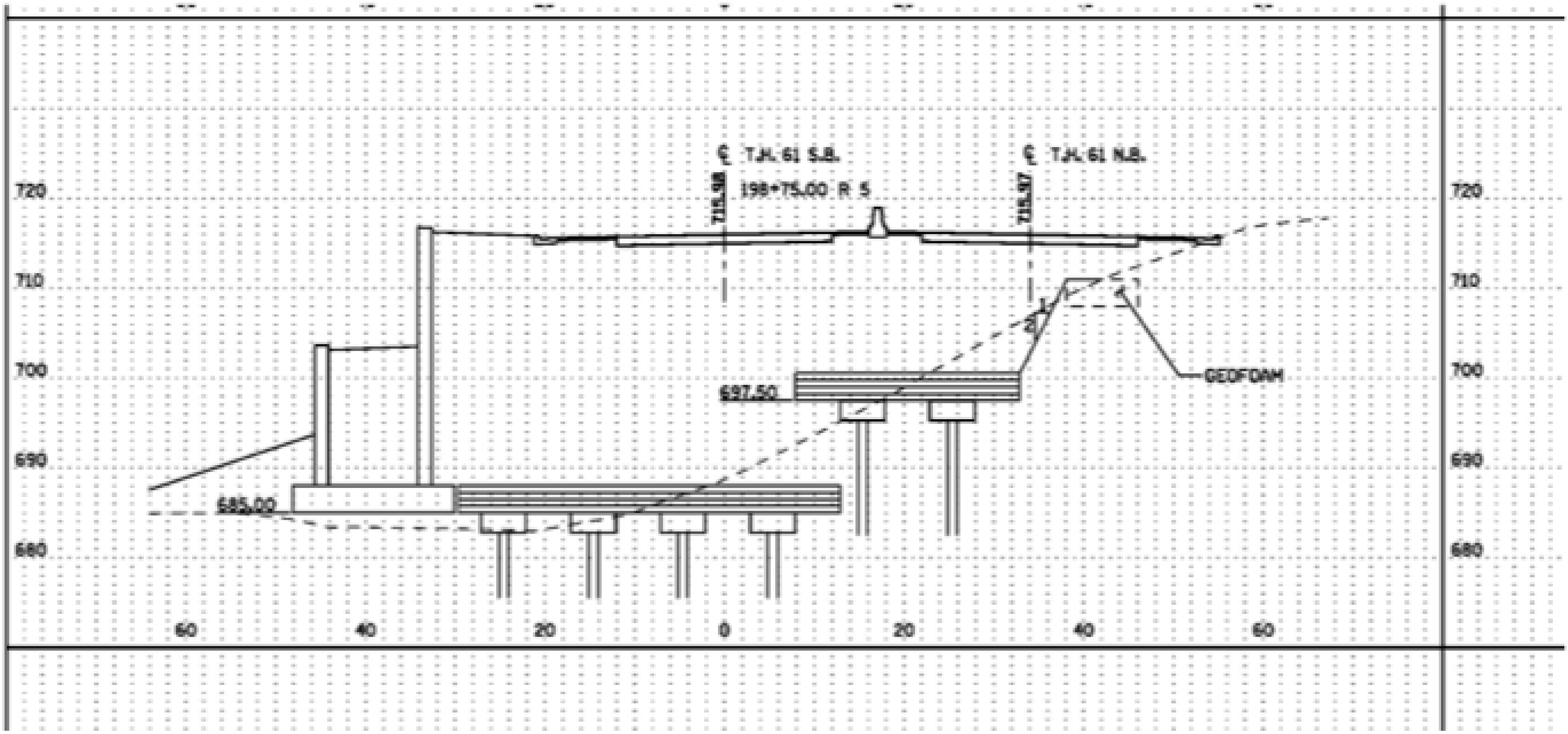
7.2.2 Ground Conditions

The general stratigraphy of the site consists of six different strata. The strata descriptions are generally summarized in Table 6-3.

Table 6-3. Brief Summary of US 61 Project Stratigraphy

Stratum	Description	N_{60} *Avg.* (blows/ft)	Depth from Existing Grade (ft)
I	Mixture of Sand, Silt and Clay	15	0 to 50
II	Slightly Organic Silty Clay Loam	9	50 to 110
III	Sand with some gravel	50	110 to 125
IV	Slightly Organic Silty Clay Loam	11	125 to 150
V	Sand with some Gravel	75+	150 to 185
VI	Bedrock	--	185+

Strata II and IV, as well as the fine-grained layers within Stratum I, exist at very high natural moisture contents and exhibit relatively low shear strength. These strata would be prone to excessive settlement under the full load of the planned embankment if settlement mitigation were not employed. These strata would also result in unacceptably low factors of safety with respect to global slope stability. Strata III and V are medium dense to very dense coarse-grained layers with moderate natural moisture contents and relatively high shear strength. The CSE columns were tipped near the top of Stratum V or approximately 155 feet beneath the existing grade prior to placement of the embankment fill. Generally, the groundwater was encountered very near the elevation of the Mississippi River which was approximately 7 feet beneath existing grade during non-flood conditions.



Minnesota DOT

Figure 6-19. Cross section of column supported approach embankment, US 61.

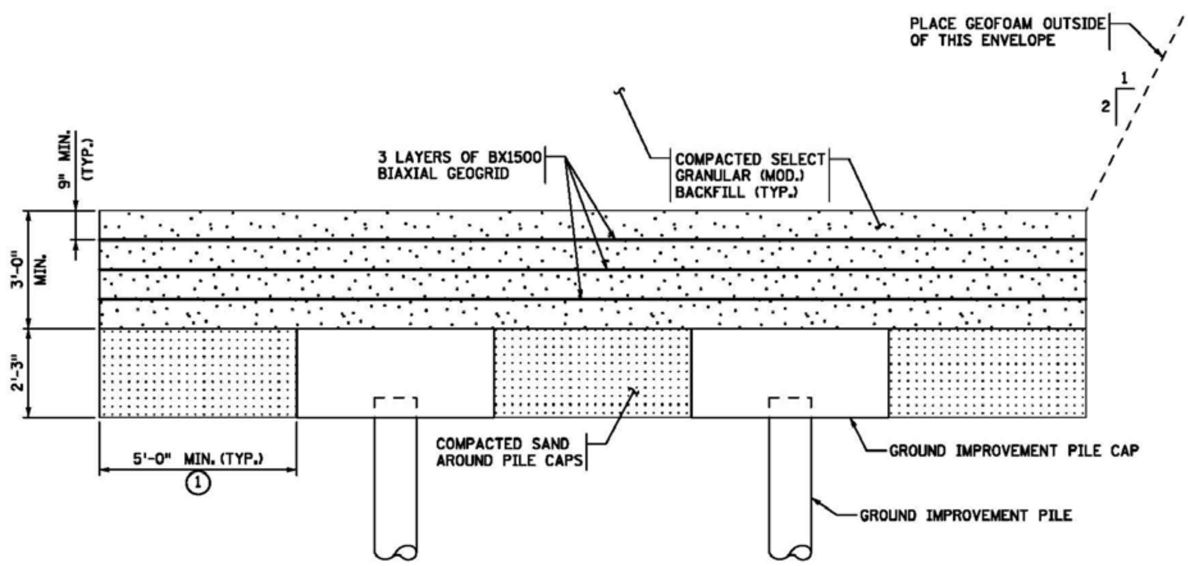
7.2.3 Ground Modification Solution

A CSE was utilized as the primary means of settlement and stability mitigation. This technology was selected to:

1. protect the construction schedule relative to pre-loading or surcharging;
2. avoid reliance on specialty sub-contractors;
3. achieve the depth of improvement necessary to satisfy the stringent performance requirements; and
4. provide a robust, reliable, and economical system commensurate with the 100-yr design life of the structure.

Other alternatives that were considered included extending the bridge, lightweight fill, pre-loading and surcharge, in situ mixing with cementitious materials, and various columns types for the CSE. Schedule, cost, reliability, and the ability to self-perform the installation lead the Contractor to choose the selected alternative.

The CSE included 12.75-inch O.D. open-ended steel pipe piles spaced on a 10-foot center-to-center square grid. The LTP placed above the columns to facilitate arching consisted of well-compacted, free-draining select granular fill reinforced with three levels of biaxial geogrid (Figure 6-20).



Minnesota DOT

Figure 6-20. Cross section of load transfer platform, US 61.

To reduce the span length and associated thickness of the LTP, 5-foot diameter reinforced concrete pile caps were placed on top of each individual pipe pile (Figure 6-21).

Minnesota DOT

Figure 6-21. Pile columns and (near) pile caps installed, US 61.

The LTP was designed in general accordance with the Collin Method (Collin 2007) and the columns were designed using a combination of the Alpha method in fine-grained soils and the Beta method where sand layers existed. The column installation criterion simply consisted of a required tip elevation since their primary purpose was to control settlement to provide serviceability. An extensive subsurface investigation is necessary when specifying a tip elevation in soils and such information was collected both pre- and post-award using traditional rotary boring combined with CPT_u soundings. The piles were easily installed with Delmag D30 and D36 open-ended diesel hammers.

Instrumentation included multiple levels of borehole-type settlement devices, piezometers, strain gages embedded in the columns, and tiltmeters mounted to the face of the adjacent retaining wall. The piezometers and strain-gage data proved very useful in monitoring the performance of the system. The tiltmeters also provided useful and reliable information. Installation of the multiple borehole settlement devices in single holes to the required depths proved very difficult and the subsequent data are of questionable quality. On future projects of similar nature, horizontally-aligned shape accelerometer array (SAA) devices are considered to be a superior option for settlement monitoring.

The completed north approach embankment is shown in Figure 6-22.

Minnesota DOT

Figure 6-22. Completed north approach embankment and approach spans, US 61.

8.0 EXAMPLE PROBLEMS

8.1 Example Problem 1

8.1.1 Problem Statement

A 20.5-foot high approach embankment is to be constructed over a 20-foot thick soft compressible clay layer. The groundwater is 3 feet below grade. Because of time constraints, a column supported embankment has been selected for the support of the embankment. The following soil properties were determined as part of the exploration program:

- Soft Clay
    - Thickness 20 feet
    - Unit weight 100 pcf
    - Undrained shear strength 250 lb/ft^2
    - Poisson's ratio 0.3
    - Effective friction angle 24 degrees
    - At rest earth pressure coefficient 0.6
    - Compression ratio 0.18
    - Recompression ratio 0.01
    - Coefficient of consolidation 0.35 ft^2/day
- Embankment Fill – Silty Sand
    - Unit weight 115 pcf
    - Poisson's ratio 0.33
    - Effective friction angle 30 degrees
    - Active earth pressure coefficient 0.33
    - Young's modulus 350,000 psf
- Bridging Layer – Dense Graded Aggregate
    - Unit weight 130 pcf
    - Effective friction angle 40 degrees
    - Young's modulus 750,000 psf

Because of ROW constraints the edges of the embankment are retained by MSE walls. The embankment width is 60 feet. The length of the reinforcement for the MSE wall is 15 feet. The general cross-section of the embankment is shown in Figure 6-23.

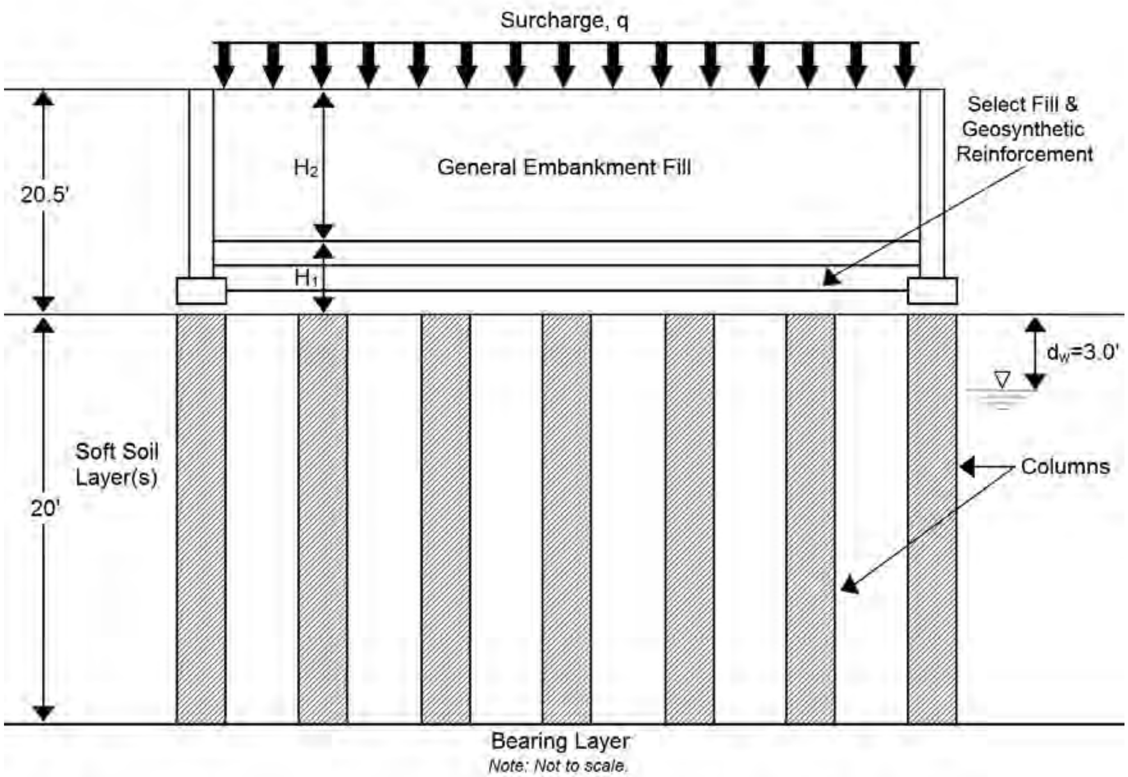


Figure 6-23. General cross-section.

Based on construction schedule, paving of the road will occur 60 days after the embankment is constructed. The maximum post pavement settlement of the embankment is 2.5 inches. The design of the columns is not included in this example as many different column types could be used, all with different design methods. The bearing layer is very stiff; therefore, for this example neither settlement of the columns into the bearing layer, nor settlement of the bearing layer in general, are considered.

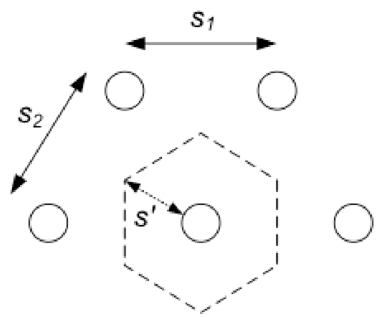
8.1.2 Solution

Step 1. Estimate Preliminary Column Spacing

Based on feasibility assessment use an area replacement ratio between 3.5 and 10%, and the clear span should be less than the embankment height divided by 1.5.

Given the embankment height of 20.5 feet, the clear span should be less than 13.7 feet but maximum recommended clear span is 10 feet.

This is the first time the State DOT has designed a CSE so they have selected a conservative triangular column spacing of 7 feet with a column top diameter of 3 feet as the initial trial spacing (see Figure 6-24).



$A_{unitcell} = s_1(s_2^2 - s_1^2/4)^{½}$

$s' = (s_2^2+2s_1^2)^{1/2}/3 - d/2$

Sloan et al. 2013

Figure 6-24. Unit cell for triangular column spacing.

Check area replacement ratio:

- Area of column = π * (D/2)2 = π * (3/2)2 = 7.07 ft

From Figure 6-24, determine the following:

- $S' = [(7^2 + 2*7^2)^{1/2} /3] – 3/2 = 2.54$ ft
- $A_{unitcell} = 7 * (7^2 – 7^2/4)^{1/2} = 42.43$ ft^2
- Area replacement ratio = 7.07/42.43 = 17% OK

Step 2. Determine Required Column Load

Effective Diameter of unit cell = D_e = 1.05 * S = 1.05 * 7 = 7.35 ft

Use Equation 6-1 to determine the column load.

Since the soft clay layer starts at grade, assume a bridging layer will be required. Assume the bridging layer is 3 feet thick. The soil used for the bridging layer has a unit weight of 130 pcf.

$Q_r = \pi(D_e/2)^2 (\gamma_e *H_2 + \gamma_{bl} * H_1 + q) = \pi (7.35/2)^2 * (115 * 17.5 + 130 * 3 + 250) = 113$ kips

Step 3. Select Preliminary Column Type

The column load is within the range of capacities of many column types listed in Table 6-1. An aggregate column, cement based column, or driven pile column could be used. For this example, assume a VCC column with a column diameter of 3 feet.

Step 4. Determine the Capacity of the Column

Both the structural and geotechnical capacity of the column must be determined. This step is not included in this design example. For column design guidance see the references listed in the chapter.

Step 5. Determine the Extent of Columns across the Embankment

Because of ROW constraints MSE walls are proposed at both sides of the embankment. The columns will therefore extend across the full width of the embankment.

Step 6. Check Critical Height Criteria

$H_{critical}$ = 1.5 * clear span = 1.5 * (2 * s') = 1.5 * (2 * 2.54) = 7.64 ft < H = 20.5 ft OK

Step 7. Determine if Bridging Layer is Required

Using the GeogridBridge spreadsheet, determine the post pavement settlement if no bridging layer is used. Screenshots are shown in Figures 6-25 and 6-26.

GeogridBridge2.0 analyzes column-supported embankments with geosynthetic-reinforced bridging layers.
The complete report by Filz and Smith (2006), plus all Main sheet comments, as well as the CGPR report by Sloan et al. (2014) should be read before using this workbook.
Provide the input data in the cells with red text. The cells in blue text are the calculated results based on the input data.
Definition sketches are provided in Figs. 1 through 6, which are located to the right.
Guidance information for material property values is provided in the pdf document to the right.
After providing all the proper input data, use the "Solve" button located at Cell B74.

Case 10 No existing sand layer and no bridging layer

	Bridging Layer Fill	Embankment Fill #2	Preload
Layer Thickness, H (ft)	0.0	20.5	0.0
Total Unit Weight, γ (pcf)	130	115	110
Friction Angle, ϕ (deg)	40	30	N/A
Lateral Earth Pressure Coefficient, K	0.75	0.75	N/A
Young's Modulus, E (psf)	750,000	350,000	N/A
Poisson's Ratio, ν	0.30	0.33	N/A

Pavement Plus Traffic Surcharge Pressure, q (psf)	250

Time Available for Consolidation, t (days)	60
Allowable Post-Construction Settlement, S_A (in.)	2.5

Depth to Groundwater, d_w (ft)	3.0
Unit Weight of Groundwater, γ_w (pcf)	62.4

	Exist Sand #1	Exist Sand #2	Clay #1	Clay #2
Layer Thickness, H (ft)	0.0	0.0	20.0	0.0
Total Unit Weight, γ (pcf)	125	125	100	100
Young's Modulus, E (psf)	300,000	250,000	N/A	N/A
Poisson's Ratio, ν	0.33	0.30	0.35	0.35
Lat. Earth Press. Coeff., K_0	0.50	0.50	0.60	0.60
Interface Frict. Angle btwn Soil and Column, δ (deg)	32	32	24	24
Compression Ratio, $C_{\varepsilon c}$	N/A	N/A	0.180	0.220
Recompression Ratio, $C_{\varepsilon r}$	N/A	N/A	0.010	0.022
Coeff. of Consol., c_v (ft^2/day)	N/A	N/A	0.35	
Initial Eff. Vert. Stress at Top of Layer, $\sigma'_{v,top}$ (psf)	N/A	N/A	0	939
Preconsol. Press. at Top of Layer, $p_{p,top}$ (psf)	N/A	N/A	375	1127
Initial Eff. Vert. Stress at Bottom of Layer, $\sigma'_{v,bot}$ (psf)	N/A	N/A	939	939
Preconsol. Press. at Bottom of Layer, $p_{p,bot}$ (psf)	N/A	N/A	1127	1127

	Biaxial Goegrid		
	Machine Direction	Cross-Machine Direction	Triaxial Geogrid
Type of Geosynthetic (use B for biaxial or T for triaxial)	B		
Stiffness of a Single Geogrid Layer (lb/ft)	40,000	40,000	14,000
Allowable Strength of a Single Geogrid Layer (lb/ft)	1,000	1,000	667
Number of Geogrid Layers	0		
Combined Geogrid Stiffness, J (lb/ft)	0	0	0
Combined Allowable Geogrid Strength, S_g (lb/ft)	0	0	0

	Pile Cap	Column
Vertical Distance from Top to Bottom of Element, H (ft)	0.0	20.0
Column Shape (use R for round and S for square)	R	R
Column Diameter or Width, d_c or a (ft)	3.0	3.0
Young's Modulus, E (psf)	6,500,000	6,500,000
Poisson's Ratio, ν	0.30	0.30
Column/Pile Cap Arrangement (use S for square/rectangular as in Fig. 5, or T for triangular as in Fig. 6)	T	
Center-to-Center Spacing, s_1 (ft)	7.0	
Center-to-Center Spacing, s_2 (ft)	7.0	

	Calc. Values	Criteria
Clear Spacing, $s - a$ (ft)	4.3	≤ 8.0
Area Replacement Ratio at Ground Surface, a_s	0.167	≥ 0.10
Bridging Layer Thickness, H_b (ft)	0.0	≥ 2.2
Total Embankment Height, $H_b + H_{embf\#2} \geq H_{crit}$ (ft)	20.5	≥ 7.2
Maximum Differential Settlement of Geogrid, d (in.)	13.3	N/A
Geogrid Strain, ε_g	#DIV/0!	≤ 0.05
Tension in a Single Geogrid Layer (lb/ft)	#DIV/0!	≤ 1,000
Combined Tension in the Geogrid Layers, T_g (lb/ft)	#DIV/0!	≤ 0
Post-Construction Embankment Settlement, S (in.)	2.9	≤ 2.5

Figure 6-25. GeogridBridge spreadsheet, Example Problem 1, no bridging layer.

GeogridBridge2.0 analyzes column-supported embankments with geosynthetic-reinforced bridging layers.
The complete report by Filz and Smith (2006), plus all Main sheet comments, as well as the CGPR report by Sloan et al. (2014) should be read before using this workbook.
Provide the input data in the cells with red text. The cells in blue text are the calculated results based on the input data.
Definition sketches are provided in Figs. 1 through 6, which are located to the right.
Guidance information for material property values is provided in the pdf document to the right.
After providing all the proper input data, use the "Solve" button located at Cell B74.

Case 10 No existing sand layer Bridging layer

	Bridging Layer Fill	Embankment Fill #2	Preload
Layer Thickness, H (ft)	3.0	17.5	0.0
Total Unit Weight, γ (pcf)	130	115	110
Friction Angle, ϕ (deg)	40	30	N/A
Lateral Earth Pressure Coefficient, K	0.75	0.75	N/A
Young's Modulus, E (psf)	750,000	350,000	N/A
Poisson's Ratio, ν	0.30	0.33	N/A

Pavement Plus Traffic Surcharge Pressure, q (psf)	250

Time Available for Consolidation, t (days)	60
Allowable Post-Construction Settlement, S_A (in.)	2.5

Depth to Groundwater, d_w (ft)	3.0
Unit Weight of Groundwater, γ_w (pcf)	62.4

	Exist Sand #1	Exist Sand #2	Clay #1	Clay #2
Layer Thickness, H (ft)	0.0	0.0	20.0	0.0
Total Unit Weight, γ (pcf)	125	125	100	100
Young's Modulus, E (psf)	300,000	250,000	N/A	N/A
Poisson's Ratio, ν	0.33	0.30	0.35	0.35
Lat. Earth Press. Coeff., K_0	0.50	0.50	0.60	0.60
Interface Frict. Angle btwn Soil and Column, δ (deg)	32	32	24	24
Compression Ratio, $C_{\varepsilon c}$	N/A	N/A	0.180	0.220
Recompression Ratio, $C_{\varepsilon r}$	N/A	N/A	0.010	0.022
Coeff. of Consol., c_v (ft^2/day)	N/A	N/A	0.35	
Initial Eff. Vert. Stress at Top of Layer, $\sigma'_{v,top}$ (psf)	N/A	N/A	0	939
Preconsol. Press. at Top of Layer, $p_{p,top}$ (psf)	N/A	N/A	375	1127
Initial Eff. Vert. Stress at Bottom of Layer, $\sigma'_{v,bot}$ (psf)	N/A	N/A	939	939
Preconsol. Press. at Bottom of Layer, $p_{p,bot}$ (psf)	N/A	N/A	1127	1127

	Biaxial Geogrid		
	Machine Direction	Cross-Machine Direction	Triaxial Geogrid
Type of Geosynthetic (use B for biaxial or T for triaxial)	B		
Stiffness of a Single Geogrid Layer (lb/ft)	24,000	24,000	14,000
Allowable Strength of a Single Geogrid Layer (lb/ft)	1,000	1,000	667
Number of Geogrid Layers	2		
Combined Geogrid Stiffness, J (lb/ft)	48,000	48,000	28,000
Combined Allowable Geogrid Strength, S_g (lb/ft)	2,000	2,000	1,334

	Pile Cap	Column
Vertical Distance from Top to Bottom of Element, H (ft)	0.0	20.0
Column Shape (use R for round and S for square)	R	R
Column Diameter or Width, d_c or a (ft)	3.0	3.0
Young's Modulus, E (psf)	6,500,000	6,500,000
Poisson's Ratio, ν	0.30	0.30
Column/Pile Cap Arrangement (use S for square/rectangular as in Fig. 5, or T for triangular as in Fig. 6)	T	
Center-to-Center Spacing, s_1 (ft)	7.0	
Center-to-Center Spacing, s_2 (ft)	7.0	

	Calc. Values	Criteria
Clear Spacing, $s - a$ (ft)	4.3	≤ 8.0
Area Replacement Ratio at Ground Surface, a_s	0.167	≥ 0.10
Bridging Layer Thickness, H_b (ft)	3.0	≥ 2.2
Total Embankment Height, $H_b + H_{embk2} \geq H_{crit}$ (ft)	20.5	≥ 7.2
Maximum Differential Settlement of Geogrid, d (in.)	2.7	N/A
Geogrid Strain, ε_g	0.003	≤ 0.05
Tension in a Single Geogrid Layer (lb/ft)	73	$\leq 1,000$
Combined Tension in the Geogrid Layers, T_g (lb/ft)	146	$\leq 2,000$
Post-Construction Embankment Settlement, S (in.)	2.5	≤ 2.5

Figure 6-26. GeogridBridge spreadsheet, Example Problem 1, with bridging layer.

The maximum calculated post pavement settlement at the surface of the embankment is 2.9 inches. The maximum allowable is 2.5 inches therefore a bridging layer is required.

Step 8. Determine the Post-construction Settlement Using a Bridging Layer

Assume a bridging layer 3 feet thick reinforced with two layers of biaxial geogrid. The geogrid has a long-term allowable strength of 1,000 lb/ft and a stiffness of 24,000 lb/ft in both directions.

The maximum post pavement settlement is 2.5 inches, which satisfies the project requirements.

Step 9. Check Lateral Extent of Columns

The columns extend to the edge of the embankment. Therefore, this is not an issue.

Step 10. Check Lateral Spreading

This check is to determine is the subgrade can provide enough lateral resistance so that the MSE wall do not slide from the internal forces from the embankment (Figure 6-27).

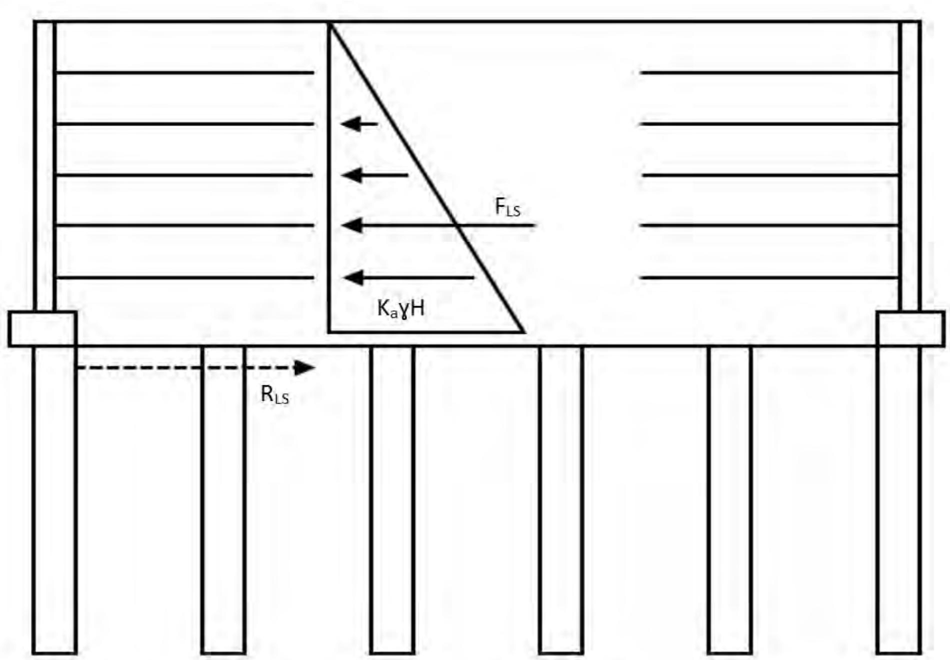


Figure 6-27. Lateral spreading.

Determine the lateral spreading force P_{Lat} from Equation 6-3.

$P_{Lat} = K_a\,(0.5\gamma H^2 + qH) = 0.33\,(0.5*115 * 20.5^2 + 250*20.5) = 9{,}665$ lb/ft

The resistance to lateral spreading must either be developed by shear at the interface between subgrade and the embankment or by adding geosynthetic reinforcement.

The resistance at subgrade is determined from Equation 6-4. The assumed length of the MSE wall reinforcement is 15 feet.

$R_{ls} = L_s * S_u = 15 * 250 = 3,750$ lb/ft

The potential for lateral spreading exists. To provide adequate resistance to lateral spreading either increase the length of the reinforcement for the MSE walls or add a lateral spreading geosynthetic to tie the two wall together. Figure 6-28 shows the lateral spreading geosynthetics used to prevent the lateral spreading of the embankment. Two layers of reinforcement are used.

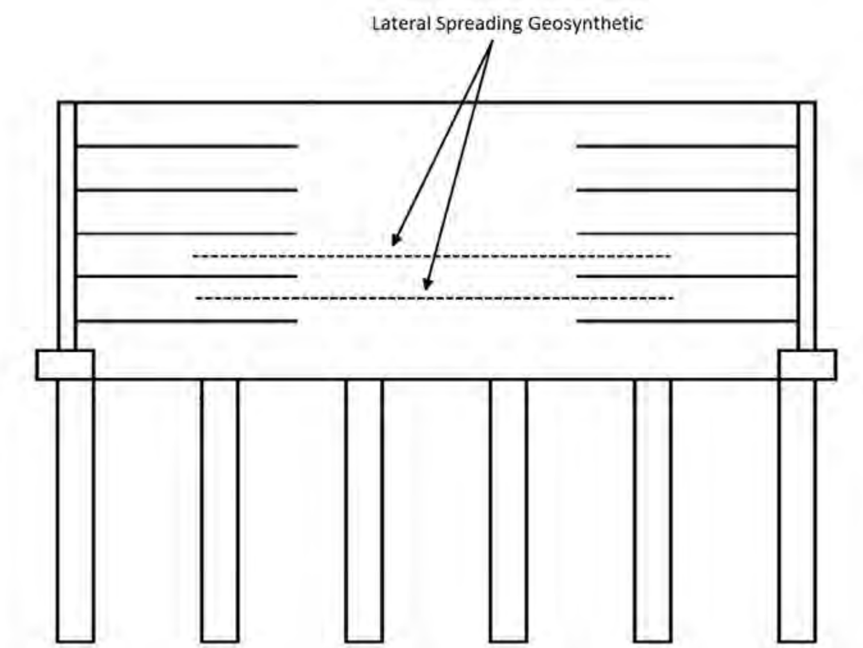


Figure 6-28. Lateral spreading geosynthetic.

Assume all of the resistance to lateral spreading is developed by the geosynthetic, thus eliminating any lateral stress at subgrade and the potential to damage the columns from the lateral spreading force.

The required long term strength of the geosynthetic reinforcement is 4,833 lb/ft (9665/2 lb/ft) for each layer.

Step 11. Determine of Overall Reinforcement Requirements

The bridging layer reinforcement is a biaxial reinforcement and the lateral spreading reinforcement is a uniaxial reinforcement as the loading is in one direction. For this reason the reinforcements will not be combined but rather kept separate.

Step 12. Check Global Stability

While it is recommended that the global stability of the CSE be evaluated, it is the author's opinion that if the CSE behaves as a true column supported embankment, there is very little potential for a global stability problem. Therefore, global stability analysis will not be performed as part of this example problem. However, if a global stability analysis is required the guidelines for the analysis are provided in the Chapter 5 Aggregate Columns.

8.2 Example Problem 2

This problem is similar to example 1. However, there is a 5 foot thick layer of sand at subgrade and then a 20 feet thick layer of soft clay. The properties of the sand layer are provided below:

- Thickness 5 ft
- Unit weight 125 pcf
- Effective friction angle 32 degrees
- Poisson's ratio 0.33
- At-rest earth pressure 0.5
- Young's modulus 300,000 psf

This problem will only analyze Step 7 to determine if a bridging layer is required.

Step 7. Determine if Bridging Layer is Required

Using the GeogridBridge spreadsheet, determine the post pavement settlement to see if the project requirements are satisfied. Screenshots are shown in Figures 6-29 and 6-30.

GeogridBridge2.0 analyzes column-supported embankments with geosynthetic-reinforced bridging layers.
The complete report by Filz and Smith (2006), plus all Main sheet comments, as well as the CGPR report by Sloan et al. (2014) should be read before using this workbook.
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After providing all the proper input data, use the "Solve" button located at Cell B74.

Case 12 Sand layer, no bridging layer

	Bridging Layer Fill	Embankment Fill #2	Preload
Layer Thickness, H (ft)	0.0	20.5	0.0
Total Unit Weight, γ (pcf)	130	115	110
Friction Angle, ϕ (deg)	40	30	N/A
Lateral Earth Pressure Coefficient, K	0.75	0.75	N/A
Young's Modulus, E (psf)	750,000	350,000	N/A
Poisson's Ratio, ν	0.30	0.33	N/A

Pavement Plus Traffic Surcharge Pressure, q (psf)	250
Time Available for Consolidation, t (days)	60
Allowable Post-Construction Settlement, S_A (in.)	2.5
Depth to Groundwater, d_w (ft)	3.0
Unit Weight of Groundwater, γ_w (pcf)	62.4

	Exist Sand #1	Exist Sand #2	Clay #1	Clay #2
Layer Thickness, H (ft)	5.0	0.0	20.0	0.0
Total Unit Weight, γ (pcf)	125	125	100	100
Young's Modulus, E (psf)	300,000	250,000	N/A	N/A
Poisson's Ratio, ν	0.33	0.30	0.35	0.35
Lat. Earth Press. Coeff., K_0	0.50	0.50	0.60	0.60
Interface Frict. Angle btwn Soil and Column, δ (deg)	32	32	24	24
Compression Ratio, $C_{\varepsilon c}$	N/A	N/A	0.180	0.220
Recompression Ratio, $C_{\varepsilon r}$	N/A	N/A	0.010	0.022
Coeff. of Consol., c_v (ft^2/day)	N/A	N/A	0.35	
Initial Eff. Vert. Stress at Top of Layer, $\sigma'_{v,top}$ (psf)	N/A	N/A	500	1252
Preconsol. Press. at Top of Layer, $p_{p,top}$ (psf)	N/A	N/A	375	1127
Initial Eff. Vert. Stress at Bottom of Layer, $\sigma'_{v,bot}$ (psf)	N/A	N/A	1252	1252
Preconsol. Press. at Bottom of Layer, $p_{p,bot}$ (psf)	N/A	N/A	1127	1127

	Biaxial Goegrid		
	Machine Direction	Cross-Machine Direction	Triaxial Geogrid
Type of Geosynthetic (use B for biaxial or T for triaxial)	B		
Stiffness of a Single Geogrid Layer (lb/ft)	24,000	24,000	14,000
Allowable Strength of a Single Geogrid Layer (lb/ft)	1,000	1,000	667
Number of Geogrid Layers	0		
Combined Geogrid Stiffness, J (lb/ft)	0	0	0
Combined Allowable Geogrid Strength, S_g (lb/ft)	0	0	0

	Pile Cap	Column
Vertical Distance from Top to Bottom of Element, H (ft)	0.0	25.0
Column Shape (use R for round and S for square)	R	R
Column Diameter or Width, d_c or a (ft)	3.0	3.0
Young's Modulus, E (psf)	6,500,000	6,500,000
Poisson's Ratio, ν	0.30	0.30
Column/Pile Cap Arrangement (use S for square/rectangular as in Fig. 5, or T for triangular as in Fig. 6)	T	
Center-to-Center Spacing, s_1 (ft)	7.0	
Center-to-Center Spacing, s_2 (ft)	7.0	

	Calc. Values	Criteria
Clear Spacing, $s - a$ (ft)	4.3	≤ 8.0
Area Replacement Ratio at Ground Surface, a_s	0.167	≥ 0.10
Bridging Layer Thickness, H_b (ft)	0.0	≥ 2.2
Total Embankment Height, $H_b + H_{emb\#2} \geq H_{crit}$ (ft)	20.5	≥ 7.2
Maximum Differential Settlement of Geogrid, d (in.)	4.5	N/A
Geogrid Strain, ε_g	#DIV/0!	≤ 0.05
Tension in a Single Geogrid Layer (lb/ft)	#DIV/0!	≤ 1,000
Combined Tension in the Geogrid Layers, T_g (lb/ft)	#DIV/0!	≤ 0
Post-Construction Embankment Settlement, S (in.)	0.9	≤ 2.5

Figure 6-29. GeogridBridge spreadsheet, Example Problem 2, no bridging layer.

GeogridBridge2.0 analyzes column-supported embankments with geosynthetic-reinforced bridging layers.
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Case 12 Sand layer, no bridging layer

	Bridging Layer Fill	Embankment Fill #2	Preload
Layer Thickness, H (ft)	0.0	20.5	0.0
Total Unit Weight, γ (pcf)	130	115	110
Friction Angle, ϕ (deg)	40	30	N/A
Lateral Earth Pressure Coefficient, K	0.75	0.75	N/A
Young's Modulus, E (psf)	750,000	350,000	N/A
Poisson's Ratio, ν	0.30	0.33	N/A

Pavement Plus Traffic Surcharge Pressure, q (psf)	250

Time Available for Consolidation, t (days)	60
Allowable Post-Construction Settlement, S_A (in.)	2.5

Depth to Groundwater, d_w (ft)	3.0
Unit Weight of Groundwater, γ_w (pcf)	62.4

	Exist Sand #1	Exist Sand #2	Clay #1	Clay #2
Layer Thickness, H (ft)	5.0	0.0	20.0	0.0
Total Unit Weight, γ (pcf)	125	125	100	100
Young's Modulus, E (psf)	300,000	250,000	N/A	N/A
Poisson's Ratio, ν	0.33	0.30	0.35	0.35
Lat. Earth Press. Coeff., K_0	0.50	0.50	0.60	0.60
Interface Frict. Angle btwn Soil and Column, δ (deg)	32	32	24	24
Compression Ratio, $C_{\varepsilon c}$	N/A	N/A	0.180	0.220
Recompression Ratio, $C_{\varepsilon r}$	N/A	N/A	0.010	0.022
Coeff. of Consol., c_v (ft^2/day)	N/A	N/A	0.35	
Initial Eff. Vert. Stress at Top of Layer, $\sigma'_{v,top}$ (psf)	N/A	N/A	500	1252
Preconsol. Press. at Top of Layer, $p_{p,top}$ (psf)	N/A	N/A	375	1127
Initial Eff. Vert. Stress at Bottom of Layer, $\sigma'_{v,bot}$ (psf)	N/A	N/A	1252	1252
Preconsol. Press. at Bottom of Layer, $p_{p,bot}$ (psf)	N/A	N/A	1127	1127

	Biaxial Goegrid		Triaxial Geogrid
	Machine Direction	Cross-Machine Direction	
Type of Geosynthetic (use B for biaxial or T for triaxial)		B	
Stiffness of a Single Geogrid Layer (lb/ft)	24,000	24,000	14,000
Allowable Strength of a Single Geogrid Layer (lb/ft)	1,000	1,000	667
Number of Geogrid Layers		0	
Combined Geogrid Stiffness, J (lb/ft)	0	0	0
Combined Allowable Geogrid Strength, S_g (lb/ft)	0	0	0

	Pile Cap	Column
Vertical Distance from Top to Bottom of Element, H (ft)	0.0	25.0
Column Shape (use R for round and S for square)	R	R
Column Diameter or Width, d_c or a (ft)	3.0	3.0
Young's Modulus, E (psf)	6,500,000	6,500,000
Poisson's Ratio, ν	0.30	0.30
Column/Pile Cap Arrangement (use S for square/rectangular as in Fig. 5, or T for triangular as in Fig. 6)	T	
Center-to-Center Spacing, s_1 (ft)	7.8	
Center-to-Center Spacing, s_2 (ft)	7.8	

	Calc. Values	Criteria
Clear Spacing, $s - a$ (ft)	5.1	≤ 8.0
Area Replacement Ratio at Ground Surface, a_s	0.136	≥ 0.10
Bridging Layer Thickness, H_b (ft)	0.0	≥ 2.5
Total Embankment Height, $H_b + H_{emb\#2} \geq H_{crit}$ (ft)	20.5	≥ 7.7
Maximum Differential Settlement of Geogrid, d (in.)	8.7	N/A
Geogrid Strain, ε_g	#DIV/0!	≤ 0.05
Tension in a Single Geogrid Layer (lb/ft)	#DIV/0!	≤ 1,000
Combined Tension in the Geogrid Layers, T_g (lb/ft)	#DIV/0!	≤ 0
Post-Construction Embankment Settlement, S (in.)	2.1	≤ 2.5

Figure 6-30. GeogridBridge spreadsheet, Example Problem 2, with bridging layer.

For a triangular column spacing of 7 feet, the post pavement settlement is 0.9 inches. The project requirement is less than or equal to 2.5 inches. Therefore, increase spacing of columns to maximize the economics of the design. For a column spacing of 7 feet 9 inches, the post pavement settlement is 2.1 inches.

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Chapter 7

DEEP MIXING AND MASS MIXING

CONTENTS

LIST OF FIGURES

1.0 DESCRIPTION AND HISTORY

1.1 Introduction

The deep mixing method and the mass mixing method both involve blending a binder with soil in situ to produce soil-cement that has improved properties, such as increased strength and reduced compressibility, compared to the untreated soil. The improved ground can be used to support embankments, retaining walls, bridge abutments, and other structures. Deep mixing has also been used to create seepage barriers, but because seepage barriers are rarely needed in transportation projects, that application is beyond the scope of this chapter.

Binder materials for the deep mixing method can consist of cement, lime, fly ash, slag, or other binder materials, as well as blends of binder materials. In current United States practice, cement and slag-cement blends are the most common types of binder. When the binder is pre-mixed with water to create a binder-water slurry that is then mixed into the ground, the process is called the :wet mixing method." When dry binder is delivered pneumatically, the process is called the "dry mixing method."

Many different types of mixing equipment have been developed, including: vertical-axis mixing equipment with multiple mixing blades mounted on one or more mixing shafts to form single columns or multiple overlapping columns from a single machine set-up location; cutter-type mixing equipment with blades or teeth mounted on two wheels rotating in opposite directions about horizontal axes to create rectangular-shaped elements at a single machine set-up location; "chainsaw" type mixers with cutting teeth to create continuous trenches as the track-mounted machine crawls in the direction of trench construction; and horizontally rotating, toothed drums attached to the end of an excavator stick to treat large areas to relatively shallow depth by moving the mixing drum vertically and laterally in the treatment area. For all types of mixing equipment, binder injection ports are located at or near the cutting and mixing blades or teeth.

Deep mixing and mass mixing are similar technologies, without a precise distinction. In general, mass mixing differs from deep mixing in three primary respects: (1) the percentage area coverage for mass mixing is 100% or nearly 100%; (2) the design strength of the mixture for mass mixing is typically lower than for deep mixing; and (3) the depth of treatment may be less than in some deep mixing applications.

1.2 Description, Historical Overview, Focus, and Scope

1.2.1 Deep Mixing

Deep mixing can be done by the wet method (Figure 7-1) or the dry method (Figure 7-2).

Figure 7-1. Deep mixing by the wet method; insert shows double-axis mixing tool.

Figure 7-2. Deep mixing by the dry method; insert shows blades of mixing tool, with port for binder delivery on shaft.

The wet method can be implemented in coarse-grained, fine-grained, and organic soils and peat. For transportation projects, the wet method of deep mixing is generally done using vertical-axis single-shaft equipment, vertical-axis multiple-shaft equipment, and cutter soil mixing equipment with horizontally rotating cutting and mixing wheels. The dry method can be implemented in soft fine-grained soils and in organic soils and peat. The dry method of deep mixing is generally done using a vertical-axis single-column mixing tool with cutting and mixing blades near the bottom of the shaft. All of these installation methods create a vertical element at each machine set-up location, where an element consists of single cylinder, a set of overlapping cylinders, or a rectangular prism of soil-cement in the ground. It is simple and common practice to refer to the treated ground as soil-cement, regardless of the type of binder used. The deep mixed elements can be used individually, or they can be overlapped to form walls, grids, or blocks of improved ground.

Important development of the deep mixing method has occurred in Japan and Scandinavia since the 1960s. Use of deep mixing on transportation projects in the United States began in the 1990s, with more than 20 major projects completed to date. Prior to about 2010, impediments to use of deep mixing in the United States included lack of familiarity with the technology, lack of readily accessible analysis and design procedures, and concern about quality control (QC) and quality assurance (QA) methods. These factors have now changed as a result of FHWA and Army Corps of Engineers investment in research, development, and technology transfer, as well as the occurrence in the United States of international conferences with a deep mixing focus in 2003, 2012, and 2015. In the aftermath of Hurricane Katrina, several deep mixing projects were completed to improve soft ground supporting levees and floodwalls, including support of earthen levee LPV 111, which involved approximately 1.7 million cubic yards of in-place soil-cement. The deep mixing work in Louisiana afforded the opportunity to develop familiarity with deep mixing in the geotechnical engineering profession, improve analysis/design procedures, and establish robust QC/QA procedures. The Louisiana experience, in combinations with previous research and development sponsored by FHWA, led to development of FHWA's deep mixing manual, which was published in 2013. Deep mixing is also now commonly used in the United States for remediation of dams (FHWA 2013). Although dams are not a transportation application, they do share important design issues with transportation embankments (settlement and stability control), and use of deep mixing for dams has further enhanced familiarity of the geotechnical engineering profession with design, construction, specifications, and QC/QA for deep mixing.

Because the FHWA (2013) deep mixing manual together with the other primary references listed below provide the information necessary for design and construction of deep mixing support systems for transportation applications, this chapter provides a summary and

overview of feasibility considerations, construction and materials, design, specifications, QC/QA, and costs.

1.2.2 Mass Mixing

As described above, mass mixing generally involves higher area replacement ratios (100% or nearly 100%), shallower treatment depths, and lower strengths than deep mixing. Nevertheless, mass mixing is very similar to deep mixing, and there is not always a clear distinction between the technologies. Consequently, a good understanding of deep mixing provides appropriate and necessary background for mass mixing.

Mass mixing can be done by the wet method or the dry method. Mass mixing includes “shallow soil mixing” and “mass stabilization”. Shallow soil mixing is generally done using large-diameter, single-axis, vertical-shaft mixing equipment with the wet method. Shallow soil mixing has been used in the United States for several decades for support of embankments and structures, as well as for treating contaminated ground by immobilizing contaminants.

Mass stabilization uses an excavator-mounted, horizontal axis mixing tool to improve soft soils (Figure 7-3).

Figure 7-3. Mass mixing; inset shows close-up of a mass mixing tool.

The method was pioneered in Scandinavia in the mid-1990s, and has since been used on several projects in the United States, including transportation projects. In Scandinavia, mass stabilization is generally done using the dry method, with the binder delivered pneumatically through the head of the mixing tool. In the United States, mass stabilization has been done using the dry method and the wet method. Mass stabilization is generally done to provide complete coverage of the treatment area, with stabilization being performed in a series of connected and overlapping blocks. Mass stabilization can be used to treat soils to a depth of about 20 feet.

Mass mixing has been used in combination with deep mixing to good effect on several projects by first mass mixing a platform, and then installing deep mixed elements through the platform. Depending on the strength of the mass mixed platform and the power of the deep mixing equipment, it may be necessary to pre-drill through the mass mixed platform before constructing the deep-mixed elements. The mass mixed platform can provide a working surface for further construction, and it can serve as a load transfer platform to reduce the number of deep mixed elements that would otherwise be necessary.

The FHWA (2013) deep mixing manual does not explicitly address mass mixing. The approach taken in this chapter is to advocate that the reader first become familiar with deep mixing, and then read here for important differences between mass mixing and deep mixing.

1.3 Glossary

For the sake of clear communication industry-wide, the following terminology and definitions are recommended:

Binder: Chemically reactive material (lime, cement, gypsum, blast furnace slag, fly ash, or other hardening reagents) that can be used for mixing with in situ soils, and upon setting, to strengthen the in situ soils and form soil-cement elements.

Binder content: Ratio of weight of dry binder to the dry weight of soil to be treated.

Binder factor: Ratio of weight of dry binder to volume of soil to be treated.

Binder factor in-place: Ratio of weight of dry binder to the volume of mixture, which is the volume of the soil to be treated plus the volume of the slurry for the wet method or the volume of the dry binder for the dry method.

Binder slurry: Stable colloidal mixture of water, binder, and admixtures that assists in loosening the soils for effective mixing, and upon setting, to strengthen the in situ soil.

Blade rotation number (BRN): Total number of mixing blade passes per meter (m) of vertical shaft movement. Blade rotation number has been developed for and is effective for monitoring mixing effort to produce well-mixed soil-cement by vertical-axis rotary methods. For horizontal axis cutter systems, BRN is not used, but cutter wheel rotations per meter of depth can be reported as an indicator of mixing energy. [Not applicable for chainsaw-type mixers]

Column: Pillar of treated soil produced in situ by a single installation process using a mixing tool, typically a rotating shaft with blades to make a round column. A rectangular barrette produced by twin horizontal mixing shafts can also be referred to as a column. See "element" and "wall", which are related geometric terms.

Deep mixing equipment: Deep mixing equipment with various mixing tools including single-vertical-shaft mixing tools, multiple-vertical-shaft mixing tools, horizontal rotating circular cutters, chainsaw-type cutters, etc.

Deep mixing method (DMM): In situ ground treatment in which soil is blended with cement and/or other binder materials to improve strength, permeability and/or compressibility characteristics (similar terms, some of which are proprietary, include Deep Soil Mixing, deep mixing, Cement Deep Mixing, Cement Deep Soil Mixing, soil cement mixing). Deep mixing can be distinguished from mass mixing as indicated in the definition of mass mixing.

Dry mixing: Process of mechanical disaggregation of the soil in situ and its mixing with binders with or without fillers and admixtures in dry powder form. Binders are delivered primarily on tool retrieval.

Element: This is an inclusive term that refers to a DMM element produced by one penetration and withdrawal of the mixing tools at a single equipment set up location. Thus, a column produced by a single-axis machine is an element, a set of overlapping columns produced by a single stroke of a multiple-shaft mixing tool is an element, and a rectangular barrette produced by a mixing tool with horizontal-axis rotating cutter blades is an element. A chainsaw-type mixing tool that travels as it mixes produces a continuous wall, not an element.

Mass mixing: Like deep mixing, mass mixing is an in situ ground treatment method in which soil is blended with cement and/or other binder materials to improve strength, permeability and/or compressibility. Mass mixing is typically distinguished from deep mixing by the following characteristics: 100% or nearly 100% area coverage, not more than about 30 feet deep, and a lower strength than many deep mixing applications; however, there is no precise dividing line between deep mixing and mass mixing. Large-diameter, single-

axis machines and horizontal rotating drums are frequently used for mass mixing. Mass mixing is a general term meant to include shallow soil mixing and mass stabilization.

Mixing tool: Equipment used to disaggregate the soil, and distribute and mix the binder with the soil. Consists of one or several rotating units equipped with several blades, arms, paddles with/without continuous or discontinuous flight augers; horizontal rotating cutter blades; horizontal rotating drums with teeth, or chainsaw-type cutters.

Penetration (downstroke): Stage/phase of mixing process cycle, in which the mixing tool is delivered to the appropriate depth (disaggregation phase for withdrawal injection and disaggregation and mixing for penetration injection). [Not applicable for chainsaw-type mixers.]

Penetration/retrieval speed: Vertical movement per unit time of the mixing tool during penetration or withdrawal. [Not applicable for chainsaw-type mixers, e.g., trench remixing and deep wall method (TRD).]

Restroke: Additional penetration and withdrawal cycle of the mixing tool to increase the binder content and/or the mixing energy. [Not applicable for chainsaw-type mixers.]

Retrieval: Withdrawal of mixing tool from bottom depth to the ground surface. Rotations during retrieval also impart additional mixing energy.

Rotation speed: Number of revolutions of the rotating unit(s) of the mixing tool per unit time.

Soil-cement: Product of deep mixing and mass mixing consisting of a mixture of the in situ soil and binder.

Strength: Dependent upon application, various strengths may be used to assess the quality of deep mixed material. For design, "strength" usually means shear strength, but during QC/QA, "strength" usually means unconfined compressive strength. For clarity, the intended type of strength should always be identified when using the term "strength".

Stroke: One complete cycle (penetration and withdrawal) of the mixing process.

Volume ratio: Ratio of the volume of slurry injected (in wet mixing) to the volume of soil to be treated.

Wall: Group of overlapping columns or elements arranged to form a continuous wall. Continuous walls can also be constructed using a chainsaw-type of mixing device. Walls can

be referred to as “shear walls,” “cutoff walls,” or “excavation support walls,” depending on the application. A shear wall can also be referred to as a “buttress”.

Water: Fresh water, free of deleterious substances that adversely affect the strength and mixing properties of the slurry, used to manufacture grout.

Water-to-binder ratio: Weight of water added to the dry binder divided by the weight of the dry binder. In wet mixing, the “water-to-binder ratio of the slurry” is determined from the weights of water and dry binder used to manufacture the slurry in a plant at the ground surface. In either wet or dry mixing, the “total-water-to-binder ratio” is the weight of water in the mixture divided by the weight of dry binder. For wet mixing, the “total-water-to-binder ratio” is the weight of slurry water plus the weight of soil water divided by the weight of dry binder. For dry mixing, the “total-water-to-slurry ratio” is the weight of soil water divided by the weight of dry binder.

Wet mixing: Process of mechanical disaggregation of the soil in situ and its mixing with slurry consisting of water and binders with or without fillers and admixtures.

Withdrawal (upstroke): Stage/phase of retrieval of the mixing tool in which the final mixing occurs for penetration injection and initial mixing for withdrawal injection. Disaggregation occurs during the penetration for both penetration injection and withdrawal injection. [Not applicable for chainsaw-type mixers (TRD).]

Withdrawal rate: The average up-hole retrieval rate of the mixing tool.

1.4 Primary References

Primary references for deep mixing and mass mixing in transportation applications include:

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- Kitazume, M. and Terashi, M. (2013). *The Deep Mixing Method*. CRC Press/Balkema, Leiden, The Netherlands.
- The deep mixing and mass mixing sections of the *GeotechTools* website, available at http://www.GeoTechTools.org.

2.0 DEEP MIXING

2.1 Feasibility and Considerations

2.1.1 Applications

In transportation infrastructure applications, the deep mixing method can be used to increase the strength and decrease the compressibility of soft ground for support of embankments, retaining walls, abutments, bridge piers, and other structures. In these applications, the soil-cement produced by deep mixing is used without reinforcement. Deep mixing can also be used for excavation support, typically with vertical steel reinforcement and lateral bracing or tie-back anchors.

Deep mixing by the wet method tends to be most useful for relatively large embankment projects due to mobilization costs. In such cases, it can also be economical to use deep mixing for foundation support of retaining walls and abutments. Deep mixing by the dry method tends to have lower mobilization costs, and it may be suitable for many projects, although design strengths for dry mixed materials are generally lower than for wet mixed materials. If it is necessary to penetrate dense and hard materials, wet mixing equipment is more capable than the lighter dry mixing equipment.

2.1.2 Advantages and Potential Disadvantages

2.1.2.1 Advantages

Advantages of deep mixing include:

- Increases the strength and decreases the compressibility of soft silts, clays, organics soils, and peat.
- Improves soft clay deposits more quickly than using prefabricated vertical drains with preloads and surcharging.
- Prevents liquefaction of loose sand deposits.
- Powerful wet-mixing equipment can penetrate layers of dense and strong material to treat underlying weak, loose, or compressible layers.
- Permits reduced embankment footprint and fill volume through use of steeper side slopes or vertical walls.
- The plan view arrangement of treatment, the treatment depth, and the degree of improvement to strength and stiffness can be easily adjusted to satisfy design requirements and subsurface conditions.

- Carries new loads placed adjacent to existing facilities so the new loads do not cause settlement of the existing facilities.
- High production capacity with large equipment.
- Materials are treated in situ, which can reduce disposal problems:
    - The dry method produces very little to no spoils.
    - Spoils from the wet method of deep mixing make excellent fill material.
- Stabilizes many types of contaminants.
- Can be used for dry land and marine projects.
- Economical on large projects.
- Dewatering is not necessary.
- Less noise and vibrations than from some other technologies.

2.1.2.2 Potential Disadvantages

Potential disadvantages and limitations of deep mixing include:

- The mobilization and unit costs can be higher than for other technologies, such as prefabricated vertical drains with preloading.
- Deep mixing requires familiarity of the engineer with specialized design, construction, specifications, and QC/QA practices.
- Cobbles, boulders, dense sand deposits, buried logs, and other obstructions can interfere with penetration of mixing equipment.
- Buried utilities and structures must be avoided. If buried features cannot be spanned and if treatment immediately adjacent to them is necessary, another technology, such as jet grouting, may be required.
- The wet method of deep mixing generally uses heavy equipment, which can require timber mats or other techniques to enable equipment to operate on soft ground.
- For the wet method of deep mixing, if there is not an opportunity for on-site use of the good quality fill generated by the spoils, the spoils may have to be transported off site for use on another project or to be disposed.
- Deep mixed elements are not normally installed at significant batters from vertical, although this is generally not a limitation for highway applications in which deep mixed elements are almost always installed vertically.

2.1.3 Feasibility Evaluations

2.1.3.1 Geotechnical

Deep mixing is a feasible method of ground improvement in very soft to medium stiff clays, very loose to medium dense sands, very soft to medium stiff organic soils, and peat. Powerful wet-method equipment can penetrate stiff clays and dense sands to reach underlying soft, loose, and organic deposits, but dense gravels, cobbles, boulders, logs, and other obstructions can make penetration difficult or impossible without predrilling or other pretreatment, which increase costs. Dry method equipment is typically lighter than wet method equipment, and it is not as capable of penetrating dense or hard layers.

2.1.3.2 Environmental Considerations

Cement is often used to treat contaminated ground in situ by immobilizing contaminants. Consequently, the deep mixing method may be a suitable or even a preferred method for improving the mechanical behavior of contaminated ground.

Deep mixing has been implemented in freezing conditions through use of insulated and heated slurry delivery lines, but this increases the cost. Also, consideration should be given to the potential for freeze-thaw cycles to damage the soil-cement, but temperature fluctuations are not known to be detrimental below the frost depth.

2.1.3.3 Site Conditions

For the wet method of deep mixing, space is necessary for an equipment yard, slurry batch plant, and equipment maneuvering. Comparatively little space is necessary for lighter dry mixing equipment, and no slurry plant is needed.

If near surface ground conditions are too soft to support mixing equipment, a working platform and/or timber mats or steel plates may be necessary.

High ground water levels are not problematic, other than to the extent they may contribute to the need for a working platform or other equipment support.

Buried utilities and structures can interfere with deep mixing, just as they can interfere with many other ground improvement technologies. Potential solutions include: designing the ground improvement and overlying facilities to span the buried utility, relocating the buried utility, and using jet grouting between the deep mixing and the buried utility or structure.

2.1.4 Limitations

Limitations of deep mixing include:

- Treatment depths are typically limited to about 130 feet.
- Dense or hard soils, cobbles, boulders, obstructions, and buried utilizes or structures can limit application of deep mixing.

2.1.5 Alternative Solutions (or Technologies)

Some of the most likely alternatives to deep mixing include the following, each of which should be evaluated for their own ability to achieve project objectives, as well as their advantages, potential disadvantages, and limitations:

- Prefabricated vertical drains with preloading and surcharging. This is often the least expensive option for treating compressible silts and clays, but it can take considerable time, continuing settlements may occur, and new embankment placed adjacent to existing embankment can cause settlement of the existing embankment.
- Densification by vibrating probes or deep dynamic compaction can be effective in coarse-grained soils, provided project conditions permit their use.
- Piles, aggregate columns, or vibro-concrete columns with column-supported embankments can be effective in many circumstances where deep mixing is also an option. Project size, project-specific constraints, and subsurface conditions will influence selection of the best approach.

2.2 Construction and Materials

2.2.1 Construction

Mixing methods can differ according to the following characteristics:

- Wet and dry methods.
- Vertical-axis rotary (single-axis and multiple-axis), horizontal-axis rotary (e.g., cutter wheel soil mixing and toothed-drum mixing), and vertical chain-saw type mixing.
- End delivery and shaft delivery of the binder for vertical axis mixing.
- Low, medium, and high pressure delivery of slurry.

The wet method of deep mixing requires a slurry plant, which includes storage silos for the binder, slurry mixing equipment, slurry agitation tank(s), and slurry pump(s). Quality control

of slurry preparation and delivery provides for constant proportions and controlled slurry delivery to the mixing equipment.

Wet mixing equipment is typically large and heavy, and working platforms with or without timber mats may be necessary. An excavator typically operates in support of each deep mixing rig to move timber matting and to contain and move spoils.

Slurry can be delivered during penetration or withdrawal of the mixing tool. For vertical axis mixing, the most common approach in the United States has been to inject slurry during penetration, with nozzles located near the bottom of the shaft or along the bottom level of cutting blades. In this approach, the slurry is mixed with the soil on both the downstroke and the upstroke, which increases the thoroughness of mixing. Good practice often involves double-stroking and/or dwell time at the bottom of the element to achieve thorough mixing at a location that would not otherwise receive a full complement of mixing blade passes.

For cutter wheel soil mixing using the wet method, penetration is often done using water to homogenize and increase the fluidity of the soil. Then a slurry with a relatively low water-to-cement ratio is injected during withdrawal.

The dry method of deep mixing typically employs lighter equipment that can often operate without working platforms, although timber mats may be necessary on very soft ground. The dry binder is typically stored in a track-mounted binder delivery unit, and the binder is delivered pneumatically to ports on the shaft near the mixing blades. The single-axis mixing tool of dry method equipment rotates rapidly compared to larger and heavier wet-method equipment. During penetration, the mixing tool advances using air without binder to break up the soil, and binder is delivered during withdrawal mixing.

2.2.2 Materials

The most common binder materials in United States practice are cement and slag-cement blends for both the wet and dry methods of deep mixing. Lime and lime-cement columns have also been installed using the dry method.

Potable water is typically used to make the slurry for the wet method of deep mixing.

2.3 Design Overview

A flowchart for design and construction of deep mixing projects is provided in Figure 7-4.

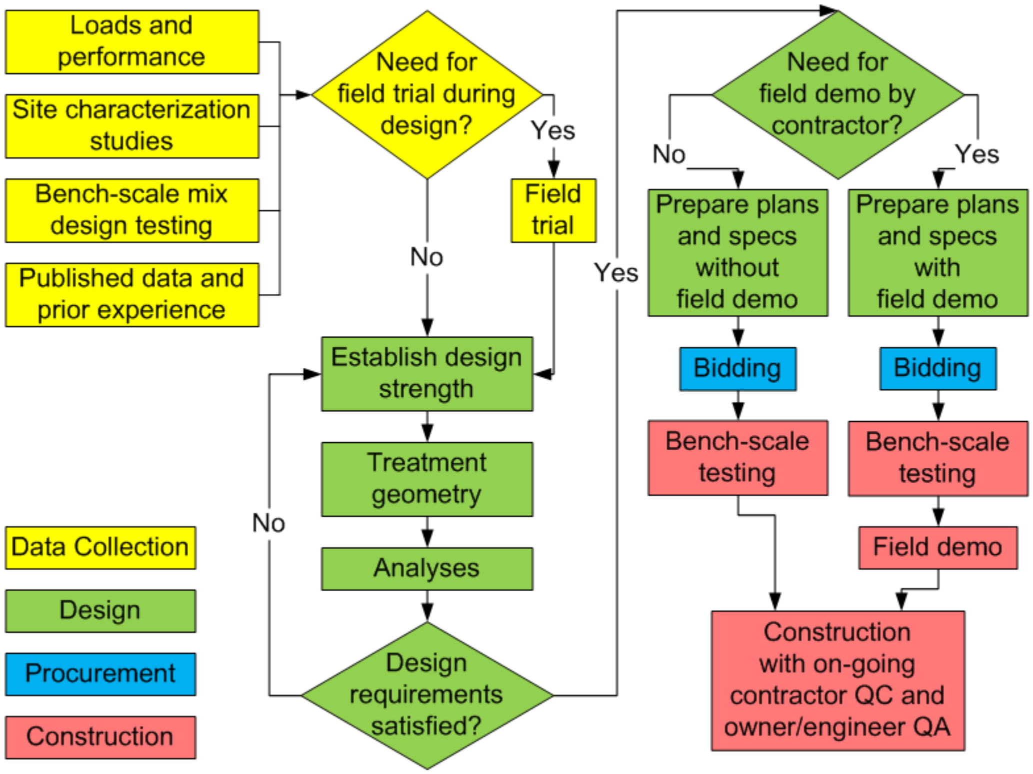


Figure 7-4. Flowchart for design and construction for DMM projects.

The key project phases are data collection, design, procurement, and construction. For present purposes, the data collection and design phases can be considered as parts of the overall design process.

2.3.1 Design Considerations

Key design considerations after the feasibility assessment discussed above include:

- Line, grade, and loading for the proposed construction.
- Desired performance in terms of settlement control and factor of safety against instability.
- Existing surface conditions, including: topography, drainage, historic and current land use, and existing utilities and facilities.
- Subsurface conditions, including:
  - Stratigraphy, with particular attention to weak layers requiring treatment.
  - Water content of soils requiring treatment. For thoroughly mixed soils, as the water content of the soil increases, the amount of binder necessary to achieve a target strength increases. However, low water content plastic clays can be

difficult to mix, but pretreatment by mixing with water instead of slurry can help improve the thoroughness of subsequent mixing with slurry.

  - Organic content and type of organics. The organic colloids in finely divided organics can interfere with cementitious reactions, whereas fibrous components of organic soils and peat do not interfere with cementitious reactions to the same extent. Experience shows that slag-cement blends tend to produce stronger mixtures than pure cement at the same overall binder factor in organic soils.
  - The presences of buried logs, stumps, and other obstructions.
  - The depth and the variability of the depth to a bearing layer, and the consistency of the bearing layer.

- Prior experience with deep mixing in similar soil conditions.
- Bench-scale trials during design to confirm treatability and provide contractors with information for bidding.
- Determination of whether a field trial is necessary during design (usually not) and whether demonstration elements are necessary at the beginning of construction (usually so).
- Select an appropriate configuration for the deep mixing. For embankments, this usually consists of isolated elements within the central portion of the embankment and shear walls oriented perpendicular to the centerline beneath the side slopes of the embankment.
- Perform iterative analyses to optimize the configuration and strength of the deep mixing ground improvement. Important issues include accounting for multiple failure modes and for variability of the field-mixed soil-cement.
- Develop plans and specifications.

2.3.2 Design Steps

As described in the FHWA (2013) deep mixing manual, design includes the following steps, which address the key design considerations listed above.

2.3.2.1 Step 1: Establish Project Requirements

This step includes establishing line, grade, loading, and desired performance in terms of settlement and factor of safety.

2.3.2.2 Step 2: Establish Representative Subsurface Conditions

This step includes determining the stratigraphy, soil property values, and the likelihood of encountering obstructions during field mixing. In addition to engineering properties like strength and compressibility of the untreated soil, other soil characteristics are important for estimating treatability, such as particle size distribution, Atterberg limits, water content, organic content, and whether organics are finely divided or fibrous.

2.3.2.3 Step 3: Establish Trial Soil-Cement Property Values

This step involves estimating the design strength and modulus of field-mixed soil-cement. Practically achievable strengths tend to decrease as the untreated soil water content increases, the organic content increases, and when the organics are finely divided. Thorough mixing of the soil and binder is easiest for loose sands and silty sands, more difficult for plastic clays, and it can be challenging for stiff plastic clays. Pretreatment of plastic clays by mixing with water prior to mixing with slurry can make thorough mixing easier. The soil-cement modulus can be measured in laboratory tests or estimated from correlations with unconfined compressive strength. Practically achievable strengths can be estimated from bench-scale treatability tests and experience on prior deep-mixing projects in similar materials. In addition to establishing trial values of soil-cement strength, strength variability should also be considered, and guidance for this is in the FHWA manual.

2.3.2.4 Step 4: Establish Trial Deep Mixed Geometry

This includes establishing the general layout pattern and the area replacement ratio in different zones beneath the embankment, as well as the treatment depth. A generally efficient layout of the soil-cement elements for embankment support is to use isolated elements beneath the central portion of the embankment, and to use overlapping elements to form continuous shear walls beneath the side slopes of the embankment. The shear walls are oriented perpendicular to the embankment alignment to provide stability for the side slopes.

2.3.2.5 Step 5: Evaluate Settlement

This is usually done by calculating the compression of the zone treated by deep mixing, and then adding compression of underlying soil layers. The zone treated by deep mixing is treated as a composite of the soil-cement elements and the untreated zone. Compression of underlying soil layers can be calculated in the same way that compression of soil under "floating" pile groups is calculated. If the embankment is low-height, a load transfer platform may be necessary (see Chapter 6).

2.3.2.6 Step 6: Evaluate Stability

In this step, it is important to evaluate all potential failure modes to ensure that adequate factor-of-safety values are achieved. Potential failure modes include: (a) sliding of the embankment above the deep mixed zone, (b) sliding beneath the deep mixed zone, (c) sliding on surfaces that pass through the deep mixed zone, (d) combined overturning stability of the shear walls and bearing capacity of the soil beneath the toe of the shear walls, (e) crushing of the toe of the shear walls when they are founded on a hard stratum, (f) shearing on vertical planes in the shear walls beneath the embankment side slopes, and (g) extrusion of soil between the parallel deep-mixed shear walls beneath the embankment side slopes. Failure modes (a) through (c) can be evaluated using a limit equilibrium slope stability analysis program, with appropriate composite shear strengths in each treated zone for failure mode (c). The FHWA deep mixing manual describes methods for analyzing failure modes (d) through (g).

The results of the analyses described in Steps 5 and 6 are compared against the performance criteria established in Step 1. If settlements are too large or factors of safety against instability are too low, the design should be made more robust by making the elements deeper, increasing the area replacement ratio(s), and/or increasing the soil-cement strength. When considering increasing the soil-cement strength, limitations on practically achievable strength for the site ground conditions should be carefully considered as discussed in Step 3. If the performance criteria are satisfied by wide margins, the element depths, area replacement ratios, and/or soil-cement strength could be decreased to optimize the design and reduce costs.

2.3.2.7 Step 7: Prepare the Plans and Specifications

Upon finalizing the design by iterating on Steps 3 through 6, the plans and specifications for construction can be prepared. These documents should be developed to allow the contractor wide latitude in means and methods, while still requiring that the specified outcomes be achieved. For example, specifying minimum area replacement ratios instead of specific element sizes and spacings is preferred, although limits on maximum spacing and minimum diameters and widths are necessary. Regarding strength, a statistically based specification should be used. For example, a typical specification for a transportation application might require that 2% of elements be cored and tested, and that 4 out of 5 specimens from each cored hole should have an unconfined compressive strength at least equal to the specified strength, and that 90% of specimens for the entire project should exhibit at least this strength. For this type of specification to be effective, the specifications also must require reports of contractor quality control using calibrated data acquisition systems for every installed element so that the owner or the owner's engineer can check for any atypical element

installations. Other requirements on geometric accuracy are also incorporated in the specifications.

2.3.3 Primary Design References

The primary references for design of deep mixing support systems for transportation applications include:

- Federal Highway Administration (FHWA). (2013). "Federal Highway Administration Design Manual: Deep Mixing for Embankment and Foundation Support." *Rep. No. FHWA-HRT-13-046*, FHWA, Washington D.C.
- FHWA. (2013). *Deep Mixing for Embankment and Foundation Support*. Authors: Bruce, M.E.C., Berg, R.R., Collin, J.G., Filz, G.M., Terashi, M. and Yang, D.S., FHWA-HRT-13-046, Federal Highway Administration, U.S.DOT, Washington D.C., 228p.
- Kitazume, M. and Terashi, M. (2013). *The Deep Mixing Method*. CRC Press/Balkema, Leiden, The Netherlands.
- The deep mixing section of the *GeotechTools* website, which is available at http://www.GeoTechTools.org.

2.4 Overview of Construction Specifications and Quality Assurance

2.4.1 Specification Development

A guide specification is available in the FHWA (2013) deep mixing report, and an updated version of this specification is available at *GeoTechTools*. All guide specifications or specifications adapted from other deep mixing projects should be very carefully reviewed and edited to appropriately address the details of each new project.

Specifications for deep mixing projects are end-result specifications, wherein the required geometry, thoroughness of mixing, and the strength of the improved ground are specified. Different contractors use different equipment and procedures such that it is neither advisable nor possible to specify means and methods. Even the plan view geometry is typically specified in a normalized fashion, such that different size elements, within a specified range, can be used to construct the ground improvement.

2.4.2 Summary of Quality Assurance

Quality assurance activities are important for most ground improvement technologies, and deep mixing is no exception. Key elements of a thorough quality assurance program include the following:

- Careful review of the contractor’s submittals, which should include:
    - Qualifications of project personnel, including an independent testing subcontractor.
    - A bench-scale mixing report. Even if the owner/engineer has completed a laboratory bench-scale mixing program and report, the contractor is generally required to perform an independent bench-scale mixing program and prepare a report. The contractor will typically focus on the mix designs most suitable for the contractor's equipment and procedures.
    - A field demonstration element report. The contractor will generally be required to construct deep mixed test elements prior to production mixing to demonstrate that the proposed mix design, mixing equipment, and mixing procedures can satisfy the specification requirements. This program allows the contractor an opportunity to try different mix designs in the field, guided by the results of bench-scale laboratory mixing and testing. Typically, all of the demonstration elements are cored from top to bottom so that the thoroughness of mixing and strength of the soil-cement can be determined. Requirements for thoroughness of mixing can be defined in the specifications in terms of percent recovery of soil-cement in each core run.
    - A deep mixing work plan, including proposed materials, equipment, mixing procedures, and element layout and identification.
    - A quality control plan, including the procedures, measurements, and documentation that will be generated to control element geometry, binder properties, mixing procedures, coring procedures, testing procedures, and element protection.
    - Daily reports, which include equipment, personnel, element construction, sampling, testing, and any problematic conditions. For each constructed element, the daily reports include element identification, element location, top and bottom elevations, and start and completion time, as well as logs versus depth of verticality, binder delivery rate, penetration/withdrawal rates, rotation rates, and bottom treatment.
    - Summary reports.

- Owner/engineer observations of materials handling, slurry preparation, slurry testing, mixing equipment, mixing procedures, coring and sampling, specimen storage, and specimen testing.
- The owner/engineer selects the elements to be cored by the contractor and the specimens for laboratory testing. The specifications include the number of elements to be cored and the number of specimens to be tested. Care should be taken to select specimens that are proportionately representative of the elements. For example, a 3-inch diameter specimen containing a 1-inch clod of unmixed soil would not be representative of a 4-foot diameter column unless the column contained a boulder-sized clod of unmixed soil. Observations of the mixing procedures, the spoils, and the overall core recovery and composition can provide the engineer with information necessary to select proportionately representative specimens for testing.
- Owner/engineer observation of specification satisfaction regarding:
    - Geometry, including element plan view dimensions, verticality, top and bottom elevations, and overlap.
    - Thoroughness of mixing. This is verified by the specified minimum core recovery percentage, after excluding untreated material.
    - Strength. Specifications typically require that strength tests satisfy statistical requirements, such as 80% of tests from an individual element exceeding a specified value for each cored element, 90% of tests for the entire project exceeding the specified value. Modern deep mixing specifications typically do not require that every test result equal or exceed a minimum strength.

2.4.3 Summary of Instrumentation, Monitoring, and Construction Control

An important part of the QC/QA philosophy for deep mixing projects is that the contractor has demonstrated suitable materials, means, and methods on validation elements that are heavily tested and shown to produce the specified outcomes for site and project conditions, all of which work is observed and documentation reviewed by the owner/engineer. The contractor then controls and documents that the same quality construction practices used on successful test elements are also used on production elements, with continued observation and document review by the owner/engineer. This process, which involves both the contractor and the owner/engineer is the primary means of quality control and quality assurance. In addition, the owner/engineer selects a limited number of production elements for full depth coring and specimens for testing. The coring and testing must be in full compliance with the project specifications. Typically, most of the selected samples from production elements are tested by the contractor's laboratory, and some are tested by the owner/engineer.

Instrumentation, monitoring, and construction control activities include:

- Calibration of all transducers, instruments, and measuring devices.
- Redundant systems for depth, rotation rate, penetration rate, and other parameters.
- Reconciliation of delivery tickets with weights of binder used in element construction.
- Weight measurements for batching dry binder and water to make slurry.
- Mud balance and Marsh funnel viscosity measurements for slurry.
- Records of geometric information, such as element plan view dimensions, location, verticality, top elevation, and bottom elevation.
- Records of binder delivery rate, penetration and withdrawal rates, rotation rates, bottom treatment, and, in some cases, power consumption, which can be used to develop a drilling index, e.g., power consumption divided by penetration rate. A drilling index can be used to indicate depth of penetration into a bearing layer for sites with variable surface of the bearing layer. Most of these quantities are presented on logs versus depth, and the logs preferably include reduced units of BRN, binder factor, etc.
- Wet-grab samples can be obtained for the contractor's information, and such samples can be particularly useful for providing an early indication of strength and strength gain with time. However, acceptance is ordinarily based on observations of full depth core recovery and UCS testing on selected core specimens. An exception is described below for mixing in coarse sands and gravels with a relatively soft soil-cement matrix.
- Core logs showing recovery and amount of unmixed or poorly mixed soil. Specifications may require recovery of at least 80% or 85% of well mixed soil-cement in each core run, where unrecovered core or poorly mixed or unmixed material is excluded from the count. The specifications may require 90% recovery of well mixed soil-cement overall for each core boring.
- For transportation projects, unconfined compression testing is often performed on 5 representative specimens selected by the owner/engineer from each full depth core boring, with 4 out of 5 specimens required to equal or exceed a specified strength, and with 90% of specimens from production elements for the entire project required to satisfy the strength requirement. Typically, no minimum strength requirement is established. Some specifications have required that a minimum average strength be achieved for each continuous sequence of, say, 20 test specimens.

- Specifications typically provide a contractor with a few standard options, should a cored production element fail to satisfy the specification requirements. For example, if it becomes apparent that a sample has failed due to an unrepresentative inclusion not visible until after the test, the owner/engineer may select a substitute sample from the same or an adjacent core run. Alternatively, the contractor may, at the contractor's own expense, drill a second boring in the same element, at a location selected by the engineer, in an attempt to demonstrate that an apparent defect is localized. The second boring is subject to the same recovery requirements and the same percentage passing the strength requirements, but the second core boring may require 10 test specimens, with the need for 8 to pass, whereas the first core boring may have only required 4 of 5 specimens to pass. If the element still fails, the contractor may propose a remediation plan subject to the approval of the owner/engineer to address all elements installed during the shift with the failed element. A successful demonstration program with continued good quality control can generally avoid this outcome.
- Optical televiewer logs can be useful in circumstances where it is difficult to recover core, such as in relatively weak soil-cement with hard particles of coarse sand and gravel. The coarse particles can become caught in the coring bit and damage the core. In such cases, the owner/engineer may allow substitution of wet-grab sampling for strength combined with optical televiewer logs for assessing thoroughness of mixing. Prior to allowing this alternate practice, all efforts to obtain quality core should be attempted. Some of the best results have been obtained with triple-tube, wire-line coring because this method tends to insulate the core from rotational forces, and the large diameter casing through which the cores are extracted reduces wobble at the core cutting and sampling depth. Several factors come into play, including the type and condition of the coring bit, the rotation rate, the penetration rate, and the fluid type and pressure.
- Protection of the deep mixed soil-cement during the curing period, any necessary trimming of the top surface without damaging the soil-cement, and protection of the exposed surface.

2.5 Cost Data

Factors that influence the cost of deep mixing projects include:

- Wet versus dry methods of deep mixing.
- The treatment volume.
- The typical and maximum element depths required.

- The specified unconfined compressive strength, which on recent projects has varied from about 100 psi to 500 psi, although strengths at the lower end of this range have been most common.
- Soil types to be treated.
- Depth of embedment into a hard bearing layer.
- Environmental conditions that may require special health and safety precautions and/or off-site disposal of spoils.
- Availability of binder materials.
- Extent of field trial testing.
- Site access.
- Mobilization/demobilization cost.
- Normal versus excessively restrictive QC/QA requirements.

2.5.1 Cost Components

According to the FHWA (2013) deep mixing manual, typical ranges of cost components can be estimated as follows: mobilization/demobilization costs for the wet method of deep mixing can be $80,000 to $150,000 per mixing rig, including support equipment, for a site located 200 miles from a qualified contractor's yard; unit costs can be in the range from $75 to $115 per cubic yard, although lower costs are sometimes encountered on large projects in competitive markets; the contractor's costs for participation in quality control and quality assurance activities can be estimated at 3% to 5% of production deep mixing costs; and engineering costs for design and construction services may be about 10% of deep mixing construction costs. However, the actual costs will be highly dependent on market conditions, project size, ground conditions, and project-specific constraints. Recent cost information includes that mobilization/demobilization costs can exceed $150,000 per mixing rig and associated support equipment.

Mobilization and unit costs for the dry method of deep mixing are typically less than for the wet method, but it is recognized that the dry method is not applicable to as wide a range of conditions or project types.

3.0 MASS MIXING

Because mass mixing and deep mixing are so closely related, without a distinct boundary between the methods, a person interested in mass mixing should first learn about deep mixing, and then read the following information and consult the mass mixing section of *GeoTechTools* to learn about mass mixing.

As mentioned previously, mass mixing is differentiated from deep mixing in these ways: (1) the percentage area coverage for mass mixing is 100% or nearly 100%, whereas the area coverage for deep mixing is typically much less than 100%, (2) the design strength of the mixture for mass mixing is typically lower than for deep mixing, and (3) the depth of treatment is typically less than in deep mixing applications. Mass mixing includes shallow soil mixing, which is done with large-diameter vertical-axis rotating shafts with mixing blades, and mass stabilization, which is done with a horizontal-axis mixing drum mounted on the stick of an excavator.

3.1 Feasibility Considerations

Mass mixing is usually intended for applications that benefit from a wide coverage at 100% or nearly 100% replacement ratio, but for which lower strengths than typically specified for deep mixing are acceptable. Uniformity of the soil-cement produced by mass mixing is operator dependent, and strength variability may be greater than for well-controlled deep mixing operations.

3.1.1 Applications

Mass mixing has proven to be effective at limiting settlement and increasing the shear strength of roadway and railway embankment foundation soils. Mass mixing has also been used to support structures like petroleum storage tanks, to stabilize excavations, in land reclamation projects, and for contaminant fixation.

Mass mixing has been used to create load transfer platforms in conjunction with deep mixed elements that carry the transferred loads to more competent foundation materials at depth, as shown in Figure 7-5. The mass mixed load transfer platform also serves as a construction working platform for installation of the deep mixed elements and other subsequent construction activity.

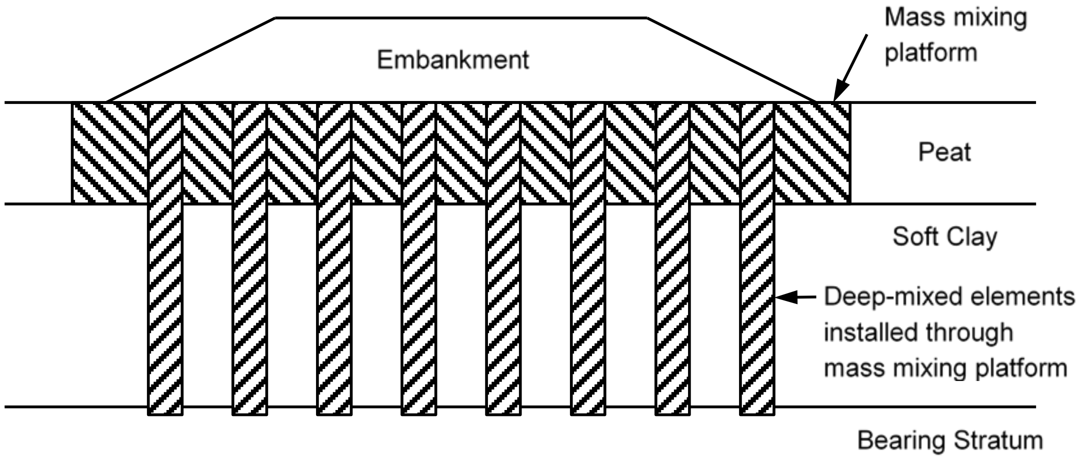


Figure 7-5. Mass mixing to create a load-transfer platform and construction working platform.

3.1.2 Advantages and Potential Disadvantages

3.1.2.1 Advantages

The principle advantages of mass mixing in comparison to deep mixing include:

- Mass mixing is typically less expensive than deep mixing on a unit volume basis, although the treatment volume per foot of depth is larger because of the larger area replacement ratio.
- Mass mixing can be done rapidly.

3.1.2.2 Potential Disadvantages

The principal disadvantages of mass mixing in comparison to deep mixing include:

- Mass mixing equipment cannot easily penetrate dense or stiff soils.
- The maximum depth of treatment for mass stabilization (mixing drum attached to backhoe stick) is limited to about 20 feet. On the other hand, the treatment depth for "shallow" soil stabilization (large-diameter, single-axis mixing equipment) can extend to 50 feet or more.
- Quality control operations, monitoring, and documentation for mass stabilization are not usually as comprehensive as for modern deep mixing in that the delivery of binder and mixing energy at every plan location and depth within a treated cell is not typically monitored or recorded, as it is for deep mixing. Instead, the total amount of binder delivered and the total mixing time in a treated cell are recorded. The quality

and uniformity of the finished product is more operator dependent for mass mixing than for deep mixing.

3.1.3 Feasibility Evaluations

3.1.3.1 Geotechnical

Mass mixing is applicable in soft organic soils and peat, soft clays, soft silt, hydraulic fill, and sludges. Mass mixing equipment is generally not designed to penetrate stiff clays or dense sands; whereas powerful wet-method deep mixing equipment can penetrate such materials. Obstructions like logs, stumps, or building debris will prevent mass mixing equipment from advancing.

3.1.3.2 Environmental Considerations

The environmental considerations for mass mixing are essentially the same as for deep mixing, although with 100% area coverage, mass mixing can stabilize contaminates over the entire treatment area.

3.1.3.3 Site Conditions

Feasibility considerations regarding site conditions for mass mixing are similar to those for deep mixing. Shallow soil mixing operations (large-diameter single-axis rotating shafts with mixing blades) using the wet method require more equipment space than mass stabilization operations (horizontal-axis mixing drum) using the dry method.

3.1.4 Limitations

Limitations of mass mixing include:

- Treatment depths are typically limited to about 50 feet for shallow soil mixing equipment and to about 20 feet for mass stabilization equipment.
- Mass mixing equipment typically cannot penetrate dense or stiff soils, cobbles, boulders, or other obstructions, and buried utilities or structures can limit application of mass mixing.

3.1.5 Alternative Solutions (or Technologies)

When poor quality soils extend to only a limited depth, excavation and replacement can be an expedient alternative to mass mixing, provided that ground water does not interfere with excavation or can be controlled.

When poor quality soils extend to greater depths, and when the comparison of alternate technologies is to the "shallow" soil mixing method, the alternative technologies previously mentioned for deep mixing can be considered.

3.2 Construction and Materials

3.2.1 Construction

3.2.1.1 Shallow Soil Mixing

Shallow soil mixing equipment generally uses relatively large-diameter (about 10 feet), single-shaft mixing equipment. Shallow soil mixing can be arranged to create a series of overlapping columns, resulting in complete coverage or nearly complete coverage. Shallow soil mixing most often uses pumped slurry, which requires a slurry plant similar to that described for the wet method of deep mixing. In very soft ground or sludges, dry binder can be delivered pneumatically. In either case, the binder is conveyed through the mixing shaft to the mixing blades. Working platforms and/or timber mats are necessary for the mixing equipment to operate on soft surfaces.

Note that shallow soil mixing is not surface stabilization, which is a method where soil is improved by blending soil and binder typically to a depth of 1 foot or less and compacting the mixed soil in lifts.

3.2.1.2 Mass Stabilization

Mass stabilization employs a horizontal-axis mixing drum attached to the stick of an excavator. Binder in slurry or dry form is delivered to the mixing drum. Contiguous rectangular cells are marked with cord or by other means, and each such cell is fully treated before moving to the next. The operator moves the rotating mixing drum vertically and horizontally to achieve treatment, typically making multiple passes through each point in the cell.

3.2.2 Materials

The materials used for mass mixing are typically the same as for deep mixing.

3.3 Design

The design process for mass mixing is similar to that for deep mixing, except that several failure modes for deep mixed systems do not need to be considered for mass mixing. Also, the strength of mass mixed soil-cement may be more variable than the strength of deep mixed soil-cement constructed using good quality control.

3.3.1 Design Considerations

Design considerations for mass mixing are similar to those for deep mixing, except that the coverage area is complete or nearly complete, so it is not necessary to make decisions about use of isolated elements, shear walls, or grid arrangements.

3.3.2 Design Procedure

The design procedure for mass mixing is similar to that for deep mixing, and the steps are listed below. Because 100% or nearly 100% area coverage is provided, it is not necessary to check for safety against vertical shearing or extrusion.

3.3.2.1 Step 1: Establish Project Requirements

This step is the same as for deep mixing.

3.3.2.2 Step 2: Establish Representative Subsurface Conditions

This step is the same as for deep mixing.

3.3.2.3 Step 3: Establish Trial Soil-Cement Property Values

This step involves the same considerations as for deep mixing. However, because the percent area coverage is large (100% or nearly 100%), lower strengths are generally used for mass mixing than for deep mixing. Also, it should be recognized at this stage that the strength of soil-cement produced by the mass mixing method is generally more variable than produced by modern deep mixing methods.

3.3.2.4 Step 4: Establish Trial Mass Mixed Geometry

For vertical-axis single-column elements, 100% coverage can be achieved using spacing-to-diameter ratios of 0.707 for a square array and 0.866 for an equilateral triangular array. The corresponding column overlap areas are 57% and 21% of the treated areas for square and triangular arrays, respectively. Thus, equilateral triangular arrays are more efficient than square arrays of columns. In many situations, it may not be necessary to achieve 100% coverage. For example, if 98% coverage is judged to be acceptable for a triangular array, the necessary spacing-to-diameter ratio is 0.929, and the column overlap areas are only 7% of the treated area.

The mass mixing geometry also includes the treatment depth. This may be important for stability and for settlement control.

3.3.2.5 Step 5: Evaluate Settlement

This is done in the same way as for deep mixing. However, a separate load transfer platform would not be necessary in a mass mixing application.

3.3.2.6 Step 6: Evaluate Stability

This step is similar to the corresponding stability evaluation for deep mixing, except that the list of potential failure modes is smaller. For mass mixing, the principal potential failure modes include: (a) sliding of the embankment above the mass mixed zone, (b) sliding beneath the mass mixed zone, (c) sliding on surfaces that pass through the mass mixed zone, (d) combined overturning stability of the mass mixed zone and bearing capacity of the soil beneath the toe of the mass mixed zone if the mass mixed zone is subjected to lateral loading, (e) crushing of the toe of the mass mixed zone if it is founded on a hard stratum and subjected to lateral loading. Failure modes (a) through (c) can be evaluated using a limit equilibrium slope stability analysis program. The FHWA deep mixing manual describes methods that can be easily adapted for analyzing failure modes (d) and (e) for mass mixed zones subjected to lateral loading.

The results of the analyses described in Steps 5 and 6 are compared against the performance criteria established in Step 1. If settlements are too large or factors of safety against instability are too low, the design should be made more robust by making the mass mixed zone larger or deeper, and/or by increasing the soil-cement strength. When considering increasing the soil-cement strength, limitations on practically achievable strength for the site ground conditions should be carefully considered as discussed in Step 3. If the performance criteria are satisfied by wide margins, the treatment volume and/or soil-cement strength could be decreased to optimize the design and reduce costs.

3.3.2.7 Step 7: Prepare the Plans and Specifications

Upon finalizing the design by iterating on Steps 3 through 6, the plans and specifications for construction can be prepared. These documents should be developed to allow the contractor wide latitude in means and methods, while still requiring that the specified outcomes be achieved. Regarding strength, a statistically based specification should be used.

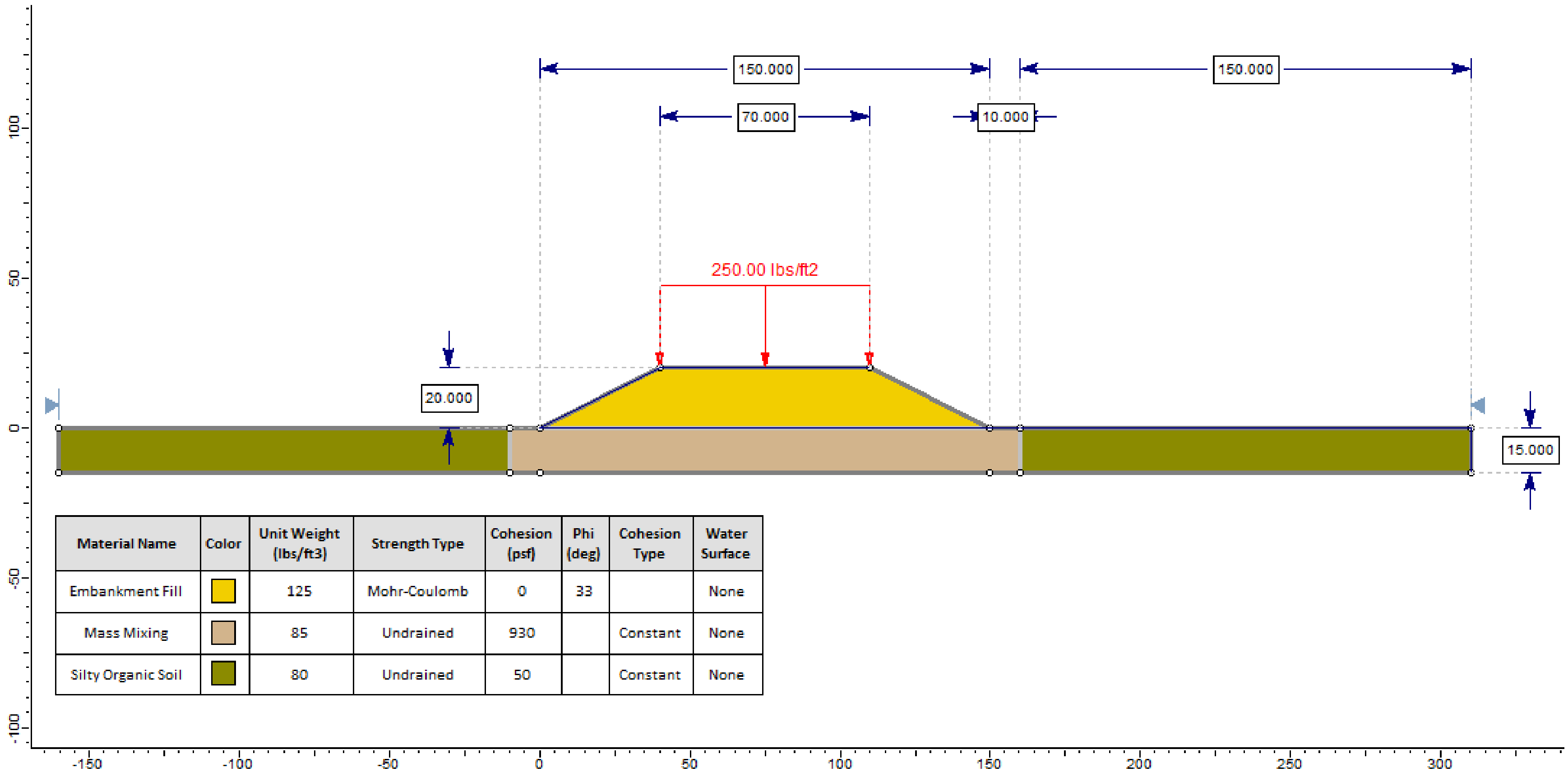
3.3.3 Design Example

3.3.3.1 Steps 1 and 2

In this example, a 20-foot high embankment is proposed with 2H:1V side slopes and a 70-foot crest width. A traffic surcharge of 250 psf is included for design. The subsurface

conditions consist of 15 feet of very soft silty organic soil overlying a hard bearing layer. The embankment and ground cross section is shown in Figure 7-6.

In Figure 7-6, the embankment soil is compacted sandy silt with a unit weight of 125 psf and a friction angle of 33 degrees. The silty organic soil has a unit weight of 80 pcf and a shear strength of 50 psf. The properties of the soil-cement are discussed in Step 3. The design requirements are that: (1) factor of safety for sliding entirely in the embankment above the soil-cement should be at least 1.3, (2) the factor of safety for sliding through the embankment, the soil-cement, and the silty organic soil beyond the soil-cement should be at least 1.5, and (3) the embankment settlement should not be more than 2 inches. The reasons for the difference in the required factor of safety values for sliding above and through the soil-cement are that the estimated friction angle of 33 degrees is conservative and more reliable than the strength estimate for the soil-cement and the consequences of failure for shallow sliding at the embankment slope surface are less than for a large sliding mass extending through the embankment, the soil-cement, and the silty organic soil beyond the soil-cement.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface
Embankment Fill		125	Mohr-Coulomb	0	33		None
Mass Mixing		85	Undrained	930		Constant	None
Silty Organic Soil		80	Undrained	50		Constant	None

Figure 7-6. Cross-section and material properties for design example.

3.3.3.2 Step 3: Establish Trial Soil-Cement Property Values

Based on review of the published literature, a design-phase laboratory testing program, and discussions with industry experts, it was determined that a field-mixed unconfined compressive strength of 26 psi (3700 psf) could be reliably and economically achieved using the dry method with a slag-cement blend and a binder factor of 8.5 pounds per cubic foot. Note that the specifications will be written around a required unconfined compressive strength and will not require any particular binder factor. This binder factor can be used, however, to estimate the slight increase in unit weight that will occur as a result of treatment by the dry method. The unit weight of the untreated silty organic soil is about 80 pcf, and treatment with 8.5 pounds per cubic foot of binder increases the unit weight to about 85 pounds per cubic foot.

For settlement calculations, the secant Young's modulus value at 50% of the unconfined compressive strength, E_{50}, can be estimated to be 150 times the unconfined compressive strength, according to the FHWA (2013) deep mixing manual, which also mentions a Poisson's ratio value, ν, of 0.1 for conditions like those described here. Using an unconfined compressive strength of 26 psi, the resulting constrained modulus value, M, is calculated as follows: E_{50} = 150(26 psi) = 3,840 psi = 550,000 psf, and $M = E_{50}\,(1 - \nu)/((1 + \nu)(1 - 2\nu))$ = (550,000 psf)(1 – 0.1)/((1 + 0.1)(1 – 2(0.1))) = 560,000 psf, which is nearly the same as the E_{50} value.

For stability calculations, the design shear strength must be obtained from the field strength. According to the FHWA (2013) deep mixing manual, four factors can be applied to the specified field strength to obtain the design shear strength: (1) a factor of 0.5 to convert from unconfined compressive strength to shear strength, (2) a factor of 0.8 to convert from peak unconfined strength to confined large-strain strength in order to provide for safety against progressive failure, (3) a curing factor, which has a value of 1.0 for a curing time of 28 days, which will be applied for this example, and (4) a variability factor to account for the relatively high variability that can occur for treated soil strength. Selecting the variability factor is described in Sections 5.4.8 and 6.1.3 of the FHWA (2013) deep mixing manual. For this example, using a factor of 1.5, a coefficient of variation of 0.6, and a probability of 70% that the actual strength of the soil-cement will equal or exceed the desired field strength, the value of the variability factor is 0.63 from Table 12 in the FHWA (2013) deep mixing manual. Appling these factors results in a design shear strength of the soil-cement, s_{sc} = (0.5)(0.8)(1.0)(0.63)(3700 psf) = 930 psf. Since 100% area coverage will be applied, there is no reduction for the area replacement ratio.

3.3.3.3 Step 4: Establish Trial Mass Mixed Geometry

The trial mass mixed geometry is shown on Figure 7-6. The mass mixing extends from the ground surface down 15 feet to the hard bearing layer and it extends 10 feet beyond the toe of the embankment.

3.3.3.4 Step 5: Evaluate Settlement

Compression of the soil-cement can be calculated by dividing the applied stress by the appropriate modulus. Near the center of the embankment, compression is approximately one-dimensional, and the constrained modulus is appropriate. Near the edge of the embankment some shear distortions will occur, but the applied load is smaller near the edge than near the center. Because the Poisson's ratio value is small, the difference between the Young's modulus and the constrained modulus is small. Using either value, the calculated compression of the soil-cement is less than 1.0 inch, e.g., (15 ft)((20 ft)(125 pcf) + 250 psf)/(550,000 psf) = 0.075 ft = 0.9 inches. If compression of the underlying bearing layer is also small, the resulting settlement will be less than the acceptable amount of 2 inches.

3.3.3.5 Step 6: Evaluate Stability

For the configuration shown in Figure 7-6, the only stability failure modes of concern are: (1) sliding of the embankment above the soil-cement zone, and (2) sliding through the embankment, the soil-cement, and the silty organic soil beyond the treated zone. For sliding in the embankment above the soil-cement zone, the lowest factor of safety is the infinite slope factor of safety, which is equal to tan(33 deg)/0.5 = 1.3, which is acceptable for that failure mode.

Stability of failure surfaces that pass through the embankment, the soil-cement, and the silty organic soil beyond the treated zone were analyzed using Spencer's method. Thorough searches for the critical circular and non-circular failures surfaces were conducted. The results are shown in Figures 7-7 and 7-8, where it can be seen that the factor of safety value for the critical circular surface is 1.55 and the factor of safety value for the critical non-circular surface is 1.50. These values are acceptable according to the criteria established in Step 1.

Because the criteria for settlement and stability are satisfied, no further iterations are necessary.

3.3.3.6 Step 7

Prepare the plans and specifications for procurement and construction.

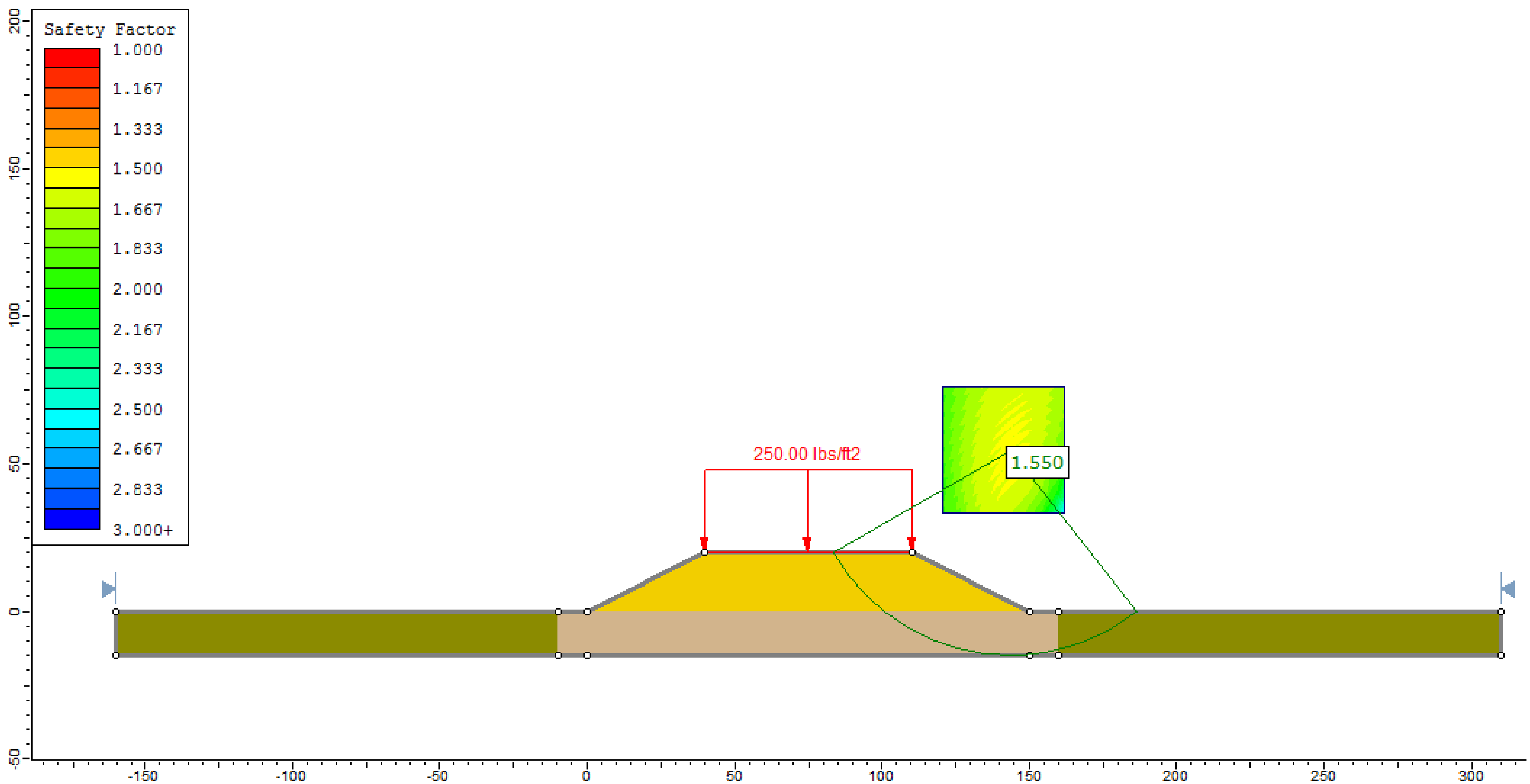


Figure 7-7. Slope stability analysis results for circular sliding surfaces.

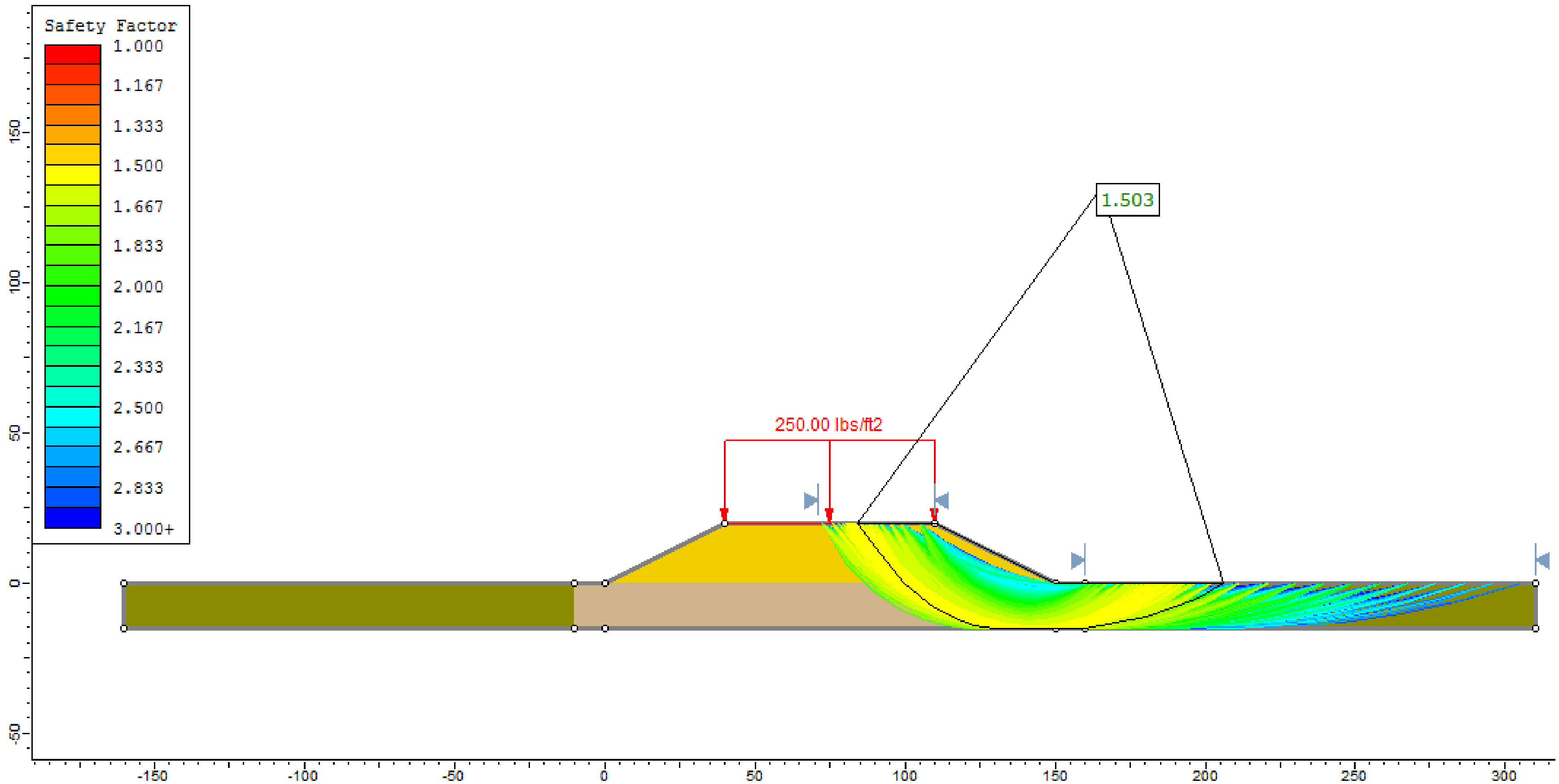


Figure 7-8. Slope stability analysis results for non-circular sliding surfaces.

3.3.4 Primary Design References

The primary references for design of deep mixing support systems for transportation applications include:

- FHWA. (2013). *Design Manual: Deep Mixing for Embankment and Foundation Support*. Authors: Bruce, M.E.C., Berg, R.R., Collin, J.G., Filz, G.M., Terashi, M. and Yang, D.S., FHWA-HRT-13-046, Federal Highway Administration, U.S. DOT, Washington D.C., 228p.
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- ALLU. (2007). *Mass Stabilisation Manual*, ALLU Finland Oy, Orimatilla. 57p. (June 24, 2014).
- The deep mixing and mass mixing sections of the *GeotechTools* website, which is available at http://www.GeoTechTools.org.

3.4 Construction Specifications and Quality Assurance

3.4.1 Specification Development

Example specifications are available at *GeoTechTools*. All specifications adapted from other mass mixing projects should be very carefully reviewed and edited to appropriately address the details of each new project. Like deep mixing, mass mixing projects use end-result specifications because of the substantial differences in contractor equipment and procedures. Specifications for mass mixing may include the following sections:

1. General
    - 1.1. Scope, project objectives, job site conditions
    - 1.2. References
    - 1.3. Qualifications
        - 1.3.1. Contractor project experience
        - 1.3.2. Contractor personnel experience
    - 1.4. Submittals
        - 1.4.1. Qualifications
        - 1.4.2. Equipment
        - 1.4.3. Materials

1.4.4. Laboratory mix design report

1.4.5. Field demonstration section plan and report

1.4.6. Work plan

1.4.7. QC/QA plan

1.4.8. Daily reports

1.4.9. Summary report

2. Materials and Equipment

2.1. Materials

2.2. Equipment

3. Execution

3.1. Laboratory mix design program

3.2. Field demonstration section

3.3. Production mixing

3.4. Quality control and quality assurance

3.4.1. Materials

3.4.2. Equipment

3.4.3. Geometry

3.4.4. Mixing process

3.4.5. Sampling and testing

3.4.6. In situ testing

3.4.7. Field load testing

3.5. Acceptance Criteria

3.5.1. Geometry

3.5.2. Consistency of Process Control

3.5.3. Unconfined Compressive Strength

3.6. Remedial Work

4. Measurement and Payment

4.1. Mobilization – lump sum

4.2. Laboratory mix design testing – lump sum

4.3. Field demonstration program – lump sum

4.4. Mixing, including QC/QA measurements, testing, documentation – price per cubic yard

3.4.2 Summary of Quality Assurance

Quality assurance activities for mass mixing are similar to those for deep mixing, except for coring and testing when the soil-cement strength is low. Although coring and core testing has been successfully accomplished on projects for which the specified unconfined compressive strengths was as low as 50 psi, in general it can be difficult to recover core for soil-cement with an unconfined compressive strength less than 100 psi, particularly if the strength is variable and if the soil-cement contains clumps of unmixed material on the scale of the core diameter. For these reasons, other types of quality assurance sampling and testing are often employed:

- If the mixture is sufficiently fluid, which it often is, wet grab sampling can be done immediately after mixing. In this technique, a sample bucket is lowered into the freshly mixed soil cement, and a door or flap is operated to collect a sample. The sample bucket is retrieved, the sample is removed, and specimens are formed in molds, similar to concrete cylinders. The cured cylinders are then tested in unconfined compression.
- Cone penetrometers can be employed, provided that the soil-cement is not too strong to prevent penetration. For very low strength mixtures, a specialized type of penetration test, called the "blade penetrometer" test in which a blade is welded to the penetrating rod, was developed in Sweden for dry-method deep-mixed columns, and it has been applied to mass mixing projects. The intent of the blade is to increase the bearing area so that more material is tested as the penetrometer is advanced, which produces less erratic results than from the smaller volume tested by a cone penetrometer.
- On some projects, full-scale embankment load tests have been constructed, and vertical and lateral movements have been monitored using settlement plates and inclinometer casings.

While coring and testing remains a preferred method, it is not practical for relatively weak soil-cement specified for some mass mixing projects, and alternative approaches like those described above are necessary. Because quality control and quality assurance is typically not as thorough for mass mixing as for deep mixing, somewhat lower values of the variability factor and/or larger factor of safety values may be warranted, although this should be evaluated on a case-by-case basis, considering all sources of uncertainty, conservatism,

consequences of failure, and desired level of performance. In the mass mixing example described above, a lower value of the variability factor was applied than would normally be used on a deep mixing project with good QC/QA.

3.4.3 Summary of Instrumentation, Monitoring, and Construction Control

Instrumentation, monitoring, and construction control for shallow soil mixing are similar to the corresponding equipment and processes for deep mixing with vertical-axis mixing equipment.

For mass stabilization with a horizontal-axis mixing drum mounted on an excavator, mixing geometry and thoroughness are operator dependent, and close observations of construction by QC and QA personnel are essential.

3.5 Cost Data

The factors that influence the cost of mass mixing are similar to those that influence the cost of deep mixing.

3.5.1 Cost Components

Mobilization costs for mass mixing can be in the range of $25,000 to $150,000, including support equipment. Unit costs can be in the range of $15 to $75 per cubic yard, with lower costs associated with large projects in competitive markets. The same types of factors and qualifications for estimating deep mixing costs also apply to estimating mass mixing costs.

3.6 Case History

A two-lane, four-span, 218-foot-long bridge was constructed to cross the Florida Power and Light discharge canal in Port Everglades, Florida, to improve truck access to a container yard and alleviate congestion on alternate routes (Gamin and Mann 2010). The approach embankments are supported with MSE walls, and the bridge is founded directly on the MSE wall fill. Design criteria included that total and differential settlements be limited to 1 inch and 0.5 inches, respectively, and that the factor-of-safety values for overall stability and bearing capacity of the MSE wall should be at least 2 and 3, respectively.

The subsurface materials at the project site consist of several feet of sand underlain by very soft organic silt, with an average silt thickness of 12 feet. The organic silt is underlain by limestone or dense cemented sand.

The dry method of mass mixing was selected to stabilize the organic silt to create a soil-cement mixture with sufficient strength and stiffness to support the MSE wall, approach

embankment fill, and bridge load. The treated zone received 100% area coverage, with treatment extending 5 feet beyond the MSE wall footings. Using this configuration, a design shear strength of the soil-cement equal to 2,160 psf satisfied the design criteria. A laboratory mix design test program using Portland cement was completed before construction to select a target weight of binder per unit volume of soil to achieve the design shear strength.

The construction process included removing the upper layer of sand and stockpiling it for later re-use. Then, a horizontal-axis mixing tool mounted on an excavator was used to blend the dry binder into the organic silt. The area to be treated was divided into adjacent cells that were each about 5 feet by 20 feet in plan view, such that 100% coverage was provided for the entire treatment area. Immediately after mixing, a geotextile was placed on top of the soil-cement mixture, and the excavated sand was replaced to provide a surcharge pressure during curing.

Construction quality was controlled by controlling the binder dose rate and mixing energy. The QC logs included identification of each cell, target binder amount, actual binder amount, and total mixing time. Quality testing was done using the blade penetrometer test with a minimum of one test per 2,500 square feet of treatment area. The blade penetration tests were supplemented by SPT borings and core drilling. To expedite construction, a relatively high binder factor was used, and the blade penetrometer tests were done one to four days after mixing. Cells that did not demonstrate the required strength within a few days after mixing were re-mixed.

Petroleum contaminated soil was encountered during construction. Because the dry method of mass mixing does not produce spoils, and because cement treatment is known to stabilize petroleum contamination, no additional work was necessary to address the contamination.

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Gamin, E. and Mann, J.A. (2010). Mass Stabilization for Environmentally Sensitive Projects in Florida. *Transportation Research Record No. 2201*, Transportation Research Board, Washington, D.C., pp. 62-69.

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The deep mixing and mass mixing sections of the *GeotechTools* website, available at http://www.GeoTechTools.org.

Chapter 8

GROUTING

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1.0 DESCRIPTION AND HISTORY

Grouting comprises a set of geotechnical techniques to introduce materials with pressure, having the objective of waterproofing and/or altering the physical characteristics of the soil or rock formation upon setting (Kutzner 1996). From the early days of simple cement slurry injections to today's sophisticated multi-material techniques, grouting has played an important role in the construction and upgrading of transportation facilities as well as other major infrastructure such as dams, levees, and tunnels. Since the late 1990s, significant advances have been made across the suite of grouting techniques, as illustrated in the proceedings of the Conferences held in New Orleans in 2003 (ASCE 2003) and 2012 (ASCE 2012) and Orlando in 2004 (ASCE 2004).

New grouting technologies continue to be developed and existing technologies refined at an accelerating pace, while the range of applications continues to expand. With the development of newer techniques such as compensation grouting and the progressive refinement of more traditional methods such as permeation and compaction grouting, grouting now offers a viable, engineered solution to a wide range of problems, including those in transportation infrastructure.

The focus and scope of this chapter is to identify the types of geotechnical problems that can be solved by grouting and to provide the user with sufficient information to make a preliminary technical and economic evaluation. Based on that evaluation, the potential for a grouting solution may be investigated further. There is a vast body of published information on each of the types of grouting, but much of it is in a format that renders it difficult to assimilate and implement by the engineering community at large. There is, therefore, a clear need for a working guide to grouting and its applications that will provide the user with a logical basis for strategic decision making. This chapter is designed to serve as that guide. Long though this chapter is, it constitutes only an introduction, and engineers embarking upon a grouting project are strongly counseled to seek expert advice.

1.1 Description

Grouting comprises a variety of techniques that employ the injection of a range of materials into soil or rock formations via boreholes to improve their engineering properties. More specifically, grouting can be used to fill fissures and voids in rock, to fill voids between the ground and overlying structures, and to treat soils and rocks to enhance strength, density, permeability, and/or homogeneity. The type of grouting method used depends on such considerations as the project's specific requirements, the soil or rock type, and the ground's amenability to different kinds of grout. Integral components of a grouting program are a thorough geotechnical investigation to identify the site conditions and to logically guide the

choice of the grouting method and its effectiveness, real time monitoring and analysis of data permitting appropriate adjustments, and a responsive verification program.

Design of a grouting program requires a thorough subsurface investigation program to assess the need for grouting and to provide information for design and construction monitoring of the grouting program. Numerous case histories have demonstrated the necessity for thorough geological exploration prior to grouting and for continuous assessment and responsive modifications during grouting. Subsurface investigations for design of grouting have more often than not been limited by economic considerations, or a failure to recognize their importance. Investigations for grouting may include any geological or geotechnical method normally used for regional and site specific investigations, and should be of sufficient detail to eliminate major surprises. It is often overlooked, however, that every hole drilled in a project – explanatory or production, is a valuable source of information about the site and all such sources shall be studied and exploited.

Major components of the subsurface investigation for grouting include leakage potential, areal and structural geology, in-situ stress conditions, hydrogeology, geochemistry, and compatibility of in-situ and grouting materials. Grout takes, mixes, procedures and pressures are best determined or estimated by conducting a grout test program at the site to provide statistical information on overall residual permeability which can be achieved.

The principal types of geotechnical grouting are shown in Figure 8-1 and are listed below:

- Rock Grouting
  - Fissures (using High Mobility Grouts (HMG))
  - Voids (natural and artificial deposits, using Low Mobility Grouts (LMG)). Note that both HMG and LMG are particulate grouts, being cement-based.
- Soil Grouting
  - Slabjacking
  - Permeation grouting (using particulate, colloidal or solution grouts)
  - Low mobility grouting – compaction grouting, displacement grouting and bulk void filling (karstic void filling, sinkhole filling, and mineral backfilling of mines)
  - Jet (or replacement) grouting
  - Soil fracture grouting (including compensation grouting)

1.2 Historical Overview

Grouting technologies have been used in the United States since the late nineteenth century, though their invention and use in other parts of the world began much earlier. The first grouting technology to be used was fissure grouting in the 1890s, followed by chemical grouting and compaction grouting (invented in the United States) in the 1950s and soil fracture (compensation) grouting in 1990s. The historical evolution of various grouting technologies was addressed in the keynote lectures at the International Conference on Grouting (ASCE 2003). Schematics of various grouting technologies are shown in Figure 8-1.

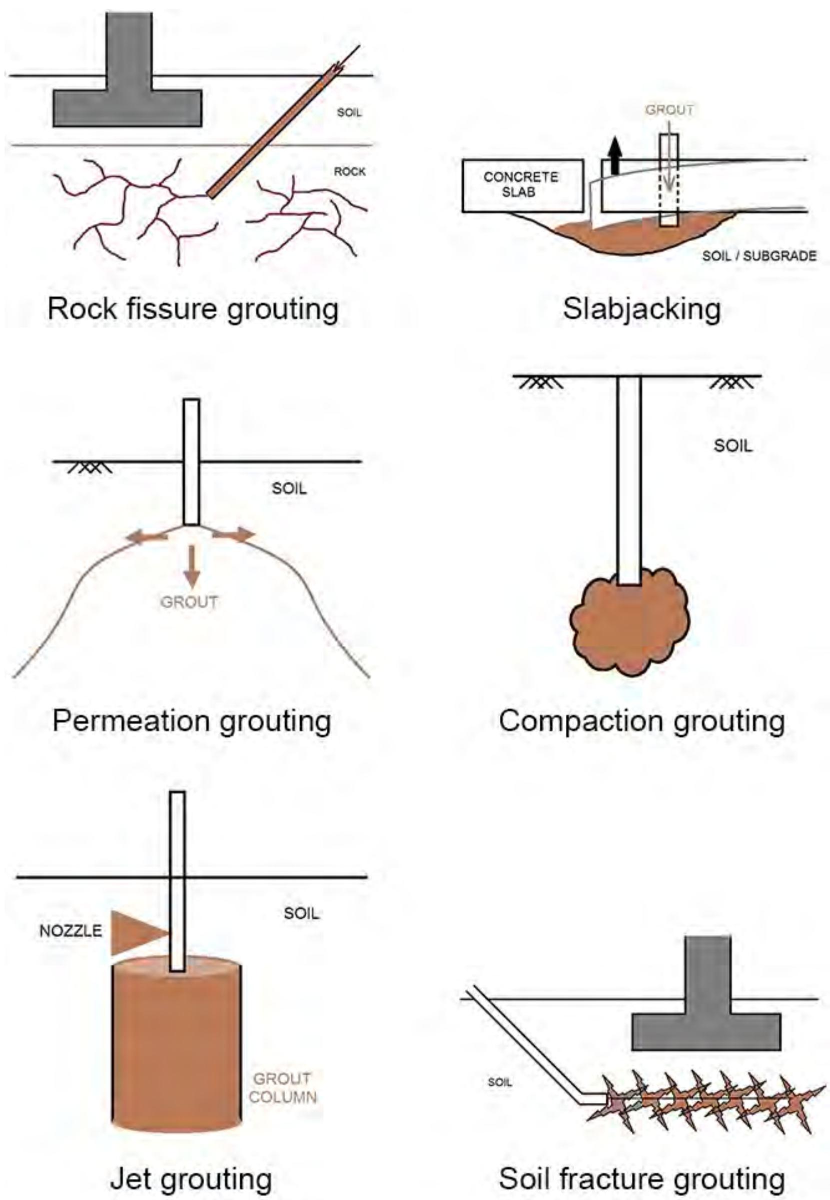

Figure 8-1. Types of grouting.

1.2.1 Rock Grouting

Rock fissure grouting is mainly used to provide hydraulic cut-offs of relatively low permeability, but it can also be used to bind together rock masses mechanically to enhance load bearing properties. Charles Berigny is credited with the invention of pressure grouting in 1802 (Houlsby 1990). This system was named the "Injection Process," and utilized excess pressure to pump a suspension of clay and lime to repair deteriorated masonry walls in the port of Dieppe, France. The earliest use of Portland cement as a grout is credited to Marc Brunel, who used it on the first Thames Tunnel in England in 1838, and to W.R. Kinnipple, who introduced the pressure injection process to England in 1856. In 1876, Thomas Hawksley used cement grouts to inject fissures in rock in England (Karol 2003).

Although W.E. Worthen is claimed to have done some masonry pier injection at Westford, Connecticut, in 1854, and R.L. Harris constructed grouted concrete foundations at Croton Lake, New York in 1891, it was not until 1893 that the pressure grouting process appears to have been used systematically to fill cavities (in limestone) under an American structure (New Croton Dam, New York) (Weaver and Bruce 2007).

This was followed by considerable activity with HMGs in repairing fissures in masonry bridge piers, and other brick and masonry structures, as well as in underwater applications (preplaced aggregate concrete and tremied foundations), many of them related to railroad construction. In 1910, grouting of Estacada Dam, Oregon, was commenced, believed by the consultants of the project to be the first systematic rock fissure grouting project to have been undertaken in the United States, with the intention of creating a hydraulic cut-off (Houlsby 1990). This proved to be the forerunner of the intense period of dam construction, and grouting, in the United States that lasted from the 1920s until the 1970s. During this time, thousands of projects were executed, largely under rigid "Prescriptive-Type" specifications to ensure standardization of approach within and between, usually federal, owner organizations. This goal was achieved, but at the expense of native innovation and in the absence of foreign input.

As a result, by the early 1980s, American practice was certainly different from, and arguably somewhat behind, European and Japanese practice. However, since then, the activities of specialty contractors, consultants, and materials and equipment suppliers, and the ever-challenging demands placed on owners principally in the field of dam rehabilitation, have resulted in significant changes. The resulting technical enhancements in techniques and abilities have been fostered by a growing use of "Performance-Type," Design-Build specifications, such as are more common in other countries, and a better understanding of the basic engineering design rationales (Baker 1985).

Recommended reading in rock grouting includes:

- ASCE. (1982). Grouting in Geotechnical Engineering. *Proc. Conference on Grouting in Geotechnical Engineering*, New Orleans, LA, Baker, W. H., Editor, ASCE, New York, NY, 2 Volumes.
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1.2.2 Slabjacking

The term "slabjacking" (or "mudjacking") is a subset of void-filling operations and refers to the pressure injection of slurry grouts of varying consistencies for the purpose of raising and re-leveling settled concrete pavement or concrete slabs. Slabjacking is also used for under-slab void filling and joint "pumping." There is no one typical practice in this field; local belief in what is best for the job at hand seems to be the norm. For instance, slabjacking may utilize a variety of fillers ranging from fly ash to lime to hot asphalt, and grout consistencies ranging from very fluid to zero slump. In addition, certain proprietary processes using expanding polyurethane foams to create uplift pressures and generate movements are used. Figure 8-2 shows a schematic of the slabjacking process.

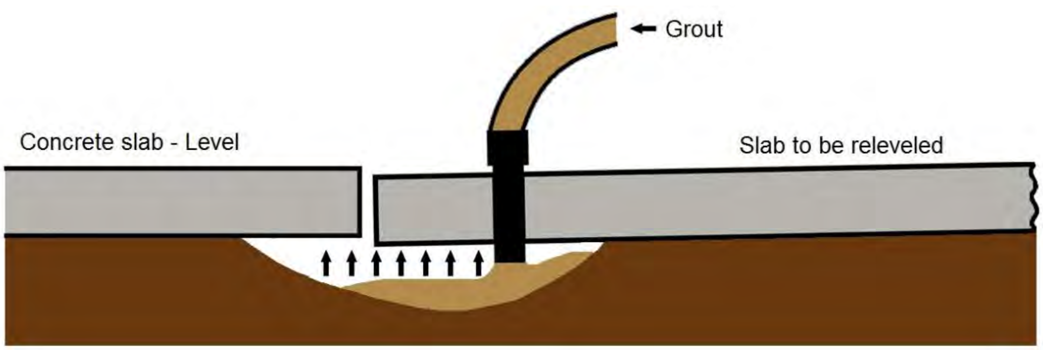


Figure 8-2. Slabjacking schematic.

1.2.3 Permeation Grouting

Permeation grouting uses materials – particulate, colloidal or solution – that can permeate soils, the exact choice largely depending on the grain size distribution (and hence, permeability) of the soil mass. Due to their relatively large particle size, conventional Portland cement (particulate) grouts can only permeate into gravels and coarse sands in properly formulated grouts. When attempting to grout finer soils, a filter cake develops at the borehole, preventing further grout permeation. Ultra-fine cement was first introduced into the United States in 1983. This led to a new family of fine-grained, fine-ground cements that could be used to permeate finer sands. This process was then taken further with the better understanding of the vital roles of pressure filtration and cohesion in controlling grout penetrability in the 1990s (Warner 1999, DePaoli et al. 1992). It is essential to understand

that the utilization of ultra-fine cement-based grouts – even if properly formulated and mixed – is alone not a guarantee of effective permeation in medium-fine sands.

Development of chemical grouting was a natural progression evolving from the limitations of early particulate grouting, such as large particle size, long setting times, instability, and poor resistance to flowing water while setting.

The first recorded patent concerning chemical grouting was obtained by Jeziorsky in 1886, and was based on injecting concentrated sodium silicate into one hole and a coagulation reagent into an adjacent hole. H.J. Joosten, a Dutch engineer, demonstrated the reliability of this chemical grouting process in 1925. His system of injecting concentrated sodium silicate during the grout pipe placement and a strong calcium chloride solution during the grout pipe withdrawal is known worldwide as the "Joosten Process." From then until the early 1950s, sodium silicate formed the basis for all chemical grouts (Karol 2003), though such "two-step" processes are now obsolete.

In the 1950s, advances in polymer chemistry, aimed at reducing the two-step Joosten process to a reliable, single-shot system (i.e., two or more chemicals mixed prior to injection into the ground) resulted in the development of a number of new, proprietary grouts. Two products—an acrylamide grout and a single-shot, silicate-based grout—dominated the American market. However, in Japan in 1974, incidents of water poisoning linked to the use of acrylamide grouts led to an immediate ban on acrylamides in that country and subsequently to a ban on all chemical grouting materials except silicate-based grouts not containing toxic additives.

At the same time in the United States, environmental pollution prevention was beginning to gain national attention. Prompted perhaps by the Japanese incident, studies were therefore conducted on acrylamide grout, while routine work continued with sodium silicate-based grouts. Responding to the concerns being voiced, the major domestic manufacturer of acrylamide grouts voluntarily withdrew the product from the market in 1978, though acrylamides had not been banned, and, in fact, are still in limited use. Because a very specialized sewer-sealing industry had grown dependent on the use of acrylamide grouts, those involved in the industry began searching for an alternative. Acrylate grouts, with properties similar to those of acrylamide grouts, but environmentally more acceptable, began to emerge as a general replacement for water control.

Sodium silicate-based grout is still the most widely used grout for soil stabilization, and indeed it was claimed even in 2003 that "virtually all construction grouting in soils in the United States is done with silicates" (Karol 2003). This situation is changing; sodium silicate is now being challenged by ultra-fine cement-based grouts due to concerns over permanency, practicality, and environmental aspects. However, in general, sodium silicate gels are still

used in “borderline” conditions where ultrafines have not yet been demonstrably effective. The silicate is reacted with either an organic or inorganic reagent, depending on the required gel properties, as described in Section 2.

Recommended reading in permeation grouting includes:

- ASCE. (1982). Grouting in Geotechnical Engineering. *Proc. Conference on Grouting in Geotechnical Engineering*, New Orleans, LA, Baker, W. H., Editor, ASCE, New York, NY, 2 Volumes.
- ASCE. (1992). Grouting, Soil Improvement and Geosynthetics. *Proc. Grouting, Soil Improvement and Geosynthetics*, Borden, R.H., Holtz, R.D., and Juran, I., Editors, Geotechnical Special Publication No. 30, ASCE, New Orleans, LA.
- ASCE. (1997). *Grouting: Compaction, Remediation, and Testing*. ASCE, Vipulanandan, C., Editor, Geotechnical Special Publication No. 66, Geo-Institute of ASCE, New York, NY, 337p.
- ASCE. (2003). Grouting and Ground Treatment. *Proc. Third International Conference*, Johnsen, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 120, Geo-Institute of ASCE, Reston, VA, 2 Vols.
- ASCE. (2012). Grouting and Deep Mixing. *Proc. Fourth International Conference*, Johnson, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 228, Geo-Institute of ASCE, New Orleans, LA.
- Bell, A.L. (1993). Jet Grouting, Chap. 7 in *Ground Improvement*, Moseley, M.P., Editor, Blackie Academic & Professional, Boca Raton, FL, pp. 149-174.
- Byle, M.J. and Borden, R.H., Editors. (1995). *Verification of Geotechnical Grouting*. Geotechnical Special Publication No. 57, ASCE, New York, NY, 177p.
- Herndon, J. and Lenahan, T. (1976). *Grouting in Soils, Vol. 1 - A State of the Art Report.* FHWA-RD-76-26 and *Vol. 2 – Design and Operations Manual*, FHWA-RD-76-27, Federal Highway Administration, U.S. DOT, Washington, D.C.
- Kutzner, C. (1996). *Grouting of Rock and Soil.* A.A. Balkema, Rotterdam, The Netherlands, 271p.
- Littlejohn, G.S. (2003). The Development of Practice in Permeation and Compensation Grouting: A Historical Review (1802 – 2002) Part 1 – Permeation Grouting. *Grouting and Ground Treatment, Proc. Third International Conference*, Johnsen, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 120, Geo-Institute of ASCE, Reston, VA, pp. 50-99.

- Tallard, G.R. and Caron, C. (1977*). Chemical Grouts for Soils*. Vol. 1 - Available Materials, FHWA-RD-77-50 and Vol. 2 - Engineering Evaluation of Available Materials, FHWA-RD-77-51, Federal Highway Administration, U.S. DOT, Washington, D.C.
- USACE. (1997). *Chemical Grouting, Technical Engineering and Design Guide*. U.S. Army Corps of Engineers, No. 24, ASCE, New York, NY.
- Waller, M.J., Huck, P.J., and Baker, W.H. (1983). *Design and Control of Chemical Grouting*, *Vol. I - Construction Control, FHWA-RD-82/036*; Krizek, R.I. and Baker, W.H., *Vol. 2 - Material Description Concepts, FHWA-RD-82/037*; Baker, W.H., *Vol. 3 - Engineering Practices, FHWA-RD-82/038;* Baker, W.H., *Vol. 4 - Executive Summary, FHWA-RD-82/039*, Federal Highway Administration, U.S. DOT, Washington, D.C.
- Yonekura, R., Terashi, M., and Shibazaki, M. (1996). Grouting and Deep Mixing. *Proc. IS-Tokyo-96- The Second International Conference on Ground Improvement Systems*, Tokyo, Japan, A.A. Balkema, Brookfield, VT, Vol. 2.

These reports and other research were instrumental in the design, specification, and utilization of chemical grouting on the Baltimore, Washington, Pittsburgh, Los Angeles, Seattle, and Boston subways.

1.2.4 Chemical Grouting

Chemical grouting is used to seal off water intrusion into tunnels or deep excavations or to pretreat relatively coarse soils when tunneling. Applications for increasing service life of facilities include stabilization of collapsible soils, reduction in liquefaction potential of soils, reduction of creep, and decreasing permeability in soils to reduce water movement in foundations. The advantages of chemical grouting are the ability to stabilize existing highway structures without traffic disruption, improvement of soils with low permeability (as low as 4×10^{-5} inches/second) up to a practical depth of 100 feet, and production of long lived facilities. Disadvantages are longer project design, construction, and monitoring durations, long setting times, relatively high costs, high toxicity of some chemicals, and break-down of grouting chemical over time thereby reducing its efficiency.

Chemical grouts are defined as any mixture of materials used for grouting purposes in which all elements of the system are pure solution with no suspended particles (Bruce 2005). These range from sodium silicate (colloidal) through true solution grouts such as polyurethanes, resins, acrylates, and lignins, to exotic materials such as precipitation grouts. Chemical grouts are very complex, expensive materials, which are typically used only in highly specialized applications involving the sealing off of water intrusions into tunnels or deep excavations, or

through abutments. An exception is the use of sodium silicate based grouts to pretreat relatively coarse soils (≤ 15% fines) for tunneling projects mostly involved with light rail or metro systems.

Sodium silicate grout was used for underpinning during the construction of the underground rail system in Baltimore, MD (Munfakh 1991). Rapid construction was achieved with minimal traffic disruption, constructed above an existing rail tunnel with performance of the system exceeding the expected values. Other case histories where chemical grouting was used include the City of Edmonton Light Rail Transit Tunnel project to achieve temporary support for tunneling for the installation of light rail system through sandy outwash (Brachman et al. 2004).

1.2.5 Compaction Grouting

Compaction grouting was pioneered on the West Coast in the 1950s, and is the only grouting technique to have its origins in the United States. It was first used to rectify structural settlements through the controlled injection of a very stiff, low mobility mix (Warner 1982). In the late 1970s, compaction grouting was introduced as a preventative, rather than a remediative, measure when the technique was used in lieu of conventional underpinning to protect surface structures from settlement during the installation of Bolton Hill Tunnel, part of the Northwest Line of the Baltimore Region Rapid Transit System (Baker et al. 1983).

The recognition that potentially liquefiable soils can be densified by compaction grouting led to test programs to verify that such loose soils beneath structures could be adequately improved by this grouting technique. The West Pinopolis Dam Test Program in 1985 showed that a compaction grouting program could be designed to obtain the level of densification required at a specific site to improve the seismic stability in-situ, provide recommendations to monitor the results, and verify the potential economics of this system (Baker 1985, Salley et al. 1987).

Since the 1980s, compaction grouting has also been used to rectify karst-related subsidence under both new and existing structures in limestone terrains (Henry 1986, Schmertmann et al. 1986) and as an integral component in the processes used to seal fast flows (Bruce et al. 2001, Bruce 2003). Compaction grouting features the use of low slump (usually 1 inch or less), low mobility grouts of high internal friction. In weak or loose soils, the grout typically forms a coherent “bulb” at the tip of the injection pipe, thus compacting and/or densifying the surrounding soil. When injected into loosened areas above tunnels or sinkholes compaction grouting will re-densify the soil and thereby prevent surficial settlement. If settlement has already occurred, careful compaction grouting may be used to lift and level any surface structures that have been impacted. Compaction grouts can be designed as an economic and

controllable medium for helping to fill large voids, even in the presence of flowing water (Bruce 1998).

Recommended reading in compaction grouting includes:

- American Society of Civil Engineers. (1982). Grouting in Geotechnical Engineering. *Proc. Conference on Grouting in Geotechnical Engineering*, New Orleans, LA, Baker, W. H., Editor, ASCE, New York, NY, 2 Volumes.
- ASCE. (1982). Grouting in Geotechnical Engineering. *Proc. Conference on Grouting in Geotechnical Engineering*, New Orleans, LA, Baker, W. H., Editor, ASCE, New York, NY, 2 Volumes.
- ASCE. (1992). Grouting, Soil Improvement and Geosynthetics. *Proc. Grouting, Soil Improvement and Geosynthetics*, Borden, R.H., Holtz, R.D., and Juran, I., Editors, Geotechnical Special Publication No. 30, ASCE, New Orleans, LA.
- ASCE. (1997). *Grouting: Compaction, Remediation, and Testing*. ASCE, Vipulanandan, C., Editor, Geotechnical Special Publication No. 66, Geo-Institute of ASCE, New York, NY, 337p.
- ASCE. (2003). Grouting and Ground Treatment. *Proc. Third International Conference*, Johnsen, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 120, Geo-Institute of ASCE, Reston, VA, 2 Vols.
- ASCE. (2012). Grouting and Deep Mixing. *Proc. Fourth International Conference*, Johnson, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 228, Geo-Institute of ASCE, New Orleans, LA.
- Baker, W.H., Cording, E.J., and MacPherson, H.H. (1983). Compaction Grouting to Control Ground Movement during Tunneling. *Underground Space - Vol. 7*, Pergamon Press Ltd.
- Bandimere, S. (1997). Compaction Grouting: State of the Practice 1997. *Grouting: Compaction, Remediation, and Testing*, Geotechnical Special Publication No. 66, Vipulanandan, Editor, Logan UT, pp. 18-31.
- Warner, J.A. (1982). Compaction Grouting – The First Thirty Years, Grouting in Geotechnical Engineering. *Grouting in Geotechnical Engineering,* Baker, W.H., Editor, ASCE, New York, NY.
- Warner, J.A. (2003). Fifty Years of Low Mobility Grouting. *Grouting and Ground Treatment, Proc. Third International Conference*, Johnsen, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 120, Geo-Institute of ASCE, New Orleans, LA, pp. 1-24.

1.2.6 Jet Grouting

Jet grouting was developed in Japan in the early 1970s (Yahiro and Yoshida 1972), based on a British concept dating from the 1960s. Since its reintroduction in Europe in Italy in the mid-1970s as a possible application for the leaning Tower of Pisa consolidation, it has been used extensively for underpinning and/or excavation support of sensitive structures, groundwater cut-off control, and tunneling applications (Welsh and Burke1991, Bell 1993, Bruce 1994, Croce et al. 2014). In the early 1980s in the United States, jet grouting utilizing conventional drilling and grouting equipment was tried on a few demonstration projects. This equipment proved to be ineffective, and jet grouting underwent a hiatus until 1986, when it was reintroduced by one specialty contractor using equipment specifically designed for the technique and incorporating contemporary European equipment and knowledge. The combination of sophisticated equipment, more extensive technical knowledge, and proper applications makes this a successful ground treatment technique, usable with almost any soil type. This is demonstrated in more than 200 successful projects completed between 1988 and 1997 alone in the United States. The rate of usage has increased substantially since then. Recent advances in jet grouting technology include the use of high efficiency tooling (monitors and nozzles) capable of producing much higher energy and consequently much larger diameters. Several such systems are available, and are known by various trade names.

Recommended reading in jet grouting includes:

- ASCE. (1982). Grouting in Geotechnical Engineering. *Proc. Conference on Grouting in Geotechnical Engineering*, New Orleans, LA, Baker, W. H., Editor, ASCE, New York, NY, 2 Volumes.
- ASCE. (1992). Grouting, Soil Improvement and Geosynthetics. *Proc. Grouting, Soil Improvement and Geosynthetics*, Borden, R.H., Holtz, R.D., and Juran, I., Editors, Geotechnical Special Publication No. 30, ASCE, New Orleans, LA.
- ASCE. (2003). Grouting and Ground Treatment. *Proc. Third International Conference*, Johnsen, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 120, Geo-Institute of ASCE, Reston, VA, 2 Vols.
- ASCE. (2012). Grouting and Deep Mixing. *Proc. Fourth International Conference*, Johnson, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 228, Geo-Institute of ASCE, New Orleans, LA.
- Bell, A.L. (1993). Jet Grouting, Chap. 7 in *Ground Improvement*, Moseley, M.P., Editor, Blackie Academic & Professional, Boca Raton, FL, pp. 149-174.

- Bruce, D.A. (1994). Jet Grouting. Chapter 8 in *Ground Control and Improvement*. Xanthakos, P.P., Abramson, L.W., and Bruce, D.A., John Wiley & Sons, New York, NY, pp. 580-679.
- Burke, G. K. (2012). The State of the Practice of Jet Grouting. *Grouting and Deep Mixing, Proc. Fourth International Conference.* Johnson, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 228, Geo-Institute of ASCE, Reston, VA., pp. 74-88.
- Croce, P., Flora, A., and Modoni, G. (2014). *Jet Grouting: Technology, Design and Control*. CRC Press, Taylor & Francis Group, Boca Raton, FL, 302p.
- Shibazaki, M. (2003). State of Practice of Jet Grouting. *Grouting and Deep Mixing, Proc. Fourth International Conference*, Johnson, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 228, Geo-Institute of ASCE Reston, VA, pp. 1-24.
- Welsh, J.P. and Burke, G.K. (1991). Jet Grouting – Uses for Soil Improvement. *Geotechnical Engineering Congress*, McLean F.G., Campbell, D.A., and Harris D.W., Editors, Geotechnical Special Publication No. 27, Geo-Institute of ASCE, Reston, VA, pp. 334-345.

1.2.7 Soil Fracture Grouting

Soil fracture grouting was introduced in the United States in the early 1990s. Its primary use is to raise settled or settling structures to their original elevation in a highly controlled manner and increase the load support characteristics of soft and/or loose soils. Soil fracture grouting works best in soils that are not free draining, but it can be applied to all soil types.

The applications of soil fracture grouting include the following:

- Raising settled structures – Soil fracture grouting has the ability to raise sensitive structures that have undergone settlement with a high degree of control, coupled with state-of-the-art instrumentation.
- Settlement control – Settlement of structures can be controlled using soil fracture grouting using predesigned fracture injections of particulate slurries. It can be used to re-level structures founded on soft, cohesive soils, or to maintain structures during tunneling, in which case it is referred to as “compensation grouting.”
- Underpinning

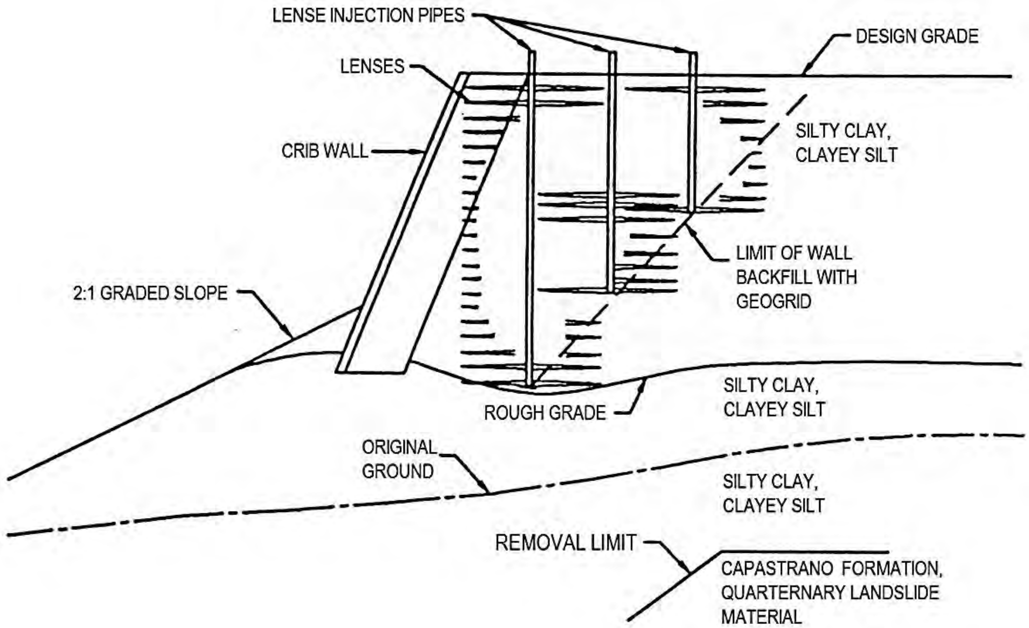
- Soil reinforcement – Fracture or lens grouting has been used to reinforce clayey soils subject to lateral movement. Fibers can be added to the grout to provide tensile strength.

The cost of soil fracture grouting cannot be accurately gauged as there is insufficient project experience in the United States. It is recommended that grouting specialists be contacted to provide feasibility and cost data for any potential project. The design of grouting programs including spacing of grouting holes, selection of grout type, and quantity and construction equipment have been presented in detail in various references (Kramer et al. 1994) and are hence not presented here.

Compensation grouting, which is a form of soil fracture grouting was used for the first time in North America for the construction of the St. Clair River tunnel, part of the Canadian National Railway System (Kramer et al. 1994). The purpose of grouting is to protect numerous sensitive above-ground structures and buried utilities during soft ground tunneling below. Additional details of the grouting project are available in Kramer et al. (1994).

In the course of routine permeation grouting activities, it was often observed that sheets or lenses of grout could be induced to travel away from the point of injection, using certain combinations of material and injection parameters. Such soil fractures could, therefore, be used to improve the overall performance of soil masses by providing a stiff "internal" grout skeleton. Developments in France in the 1970s led to the concept of using carefully controlled fracturing of the soil to compensate for surface settlements caused by underground tunneling ("claquage"). By the 1990s, "compensation grouting" or soil fracture grouting, using sophisticated construction and monitoring equipment, was being used in urban areas subject to soft ground tunneling (e.g., London's new Jubilee Line Extension and Sarnia, Ontario, Kramer et al. 1994). Most recently, the technology was applied in a similar application for the new Metro in San Juan, Puerto Rico, New York, and San Francisco.

On the West Coast, less sophisticated "lense grouting," as shown in Figure 8-3, had been undertaken for slope stabilization since the late 1980s (Chandler 1997).



Chandler 1997

Figure 8-3. Lense grouting.

Specially formulated high-rheology particulate grouts are injected repeatedly through arrays of grout pipes, the exact parameters being controlled in response to the desired surface response characteristics. Extremely careful control is exercised over the process so that the greatest benefits can be realized in terms of surface movements.

Recommended reading in soil fracture and compensation grouting includes:

- Chandler, S. C. (1997). Lense Grouting with Fiber Admixture to Reinforce Soil. *Grouting, Compaction, Remediation and Testing*, Vipulanandan, C., Editor, Geotechnical Special Publication No. 66, Geo-Institute of ASCE, Reston, VA, pp. 147-157.
- Kettle, C. (2012). Compensation Grouting – Evolution, Field of Application, and Current State of the Art in UK Practice. *Grouting and Deep Mixing, Proc. Fourth International Conference*, Johnsen, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 120, Geo-Institute of ASCE, New Orleans, LA, pp. 134-199.
- Littlejohn, G.S. (2003). The Development of Practice in Permeation and Compensation Grouting, A Historical Review, Part 2 – Compensation Grouting. *Grouting and Ground Treatment, Proc. Third International Conference*, Johnsen, L.F., Bruce, D.A., and Byle, M.J., Editors, Geotechnical Special Publication No. 120, Geo-Institute of ASCE, Reston, VA, pp. 100-144.

- Raabe, A.W. and Esters, K. (1993). Soil Fracturing Techniques for Terminating Settlements and Restoring Levels of Buildings and Structures. *Ground Improvement*, Moseley, M.P., Editor, Blackie Academic & Professional, Boca Raton, FL, pp. 175-204.

1.3 Focus and Scope

This chapter describes soil and rock grouting techniques, their applications, advantages and disadvantages, feasibility and design considerations, suitable grout materials, and costs. This chapter is organized into six different sections. Section 1 provides an historic overview of various techniques available and a glossary of various grouting terminology. Section 2 presents soil and rock grouting techniques such as permeation, low mobility (including compaction, displacement and bulk void filling – karst and sinkhole filling and mine backfilling), jet, rock fissure and other alternate grouting technologies. Further references are included for in-depth subject research, and applications of each grouting technique are illustrated with project case histories. Construction methods, drilling and grouting equipment, and grouting materials are presented in Section 3. Project planning activities are described in Section 4. Design considerations, construction methods, specifications, quality assurance, and cost data are discussed in Section 5. References used in this chapter are provided in Section 6.

1.4 Glossary (or Terminology)

There is, as yet, no internationally adopted glossary of terms relating to grouting. Word meanings and interpretations vary from country to country. The following list represents some of the standard terminology used in the United States and includes many of the definitions proposed by the ASCE Grouting Committee (Bruce 2005).

Additives: Additional grout components, such as admixtures, bentonites, mineral additives, or pozzolans, such as pulverized fly ash, blast furnace slag, and condensed silica fume.

Admixtures: An added reagent that improves the grout in a specified manner through chemical or physical action. Examples of admixtures include accelerators, air-entraining agents, anti-freezing agents, dispersants, foam agents, plasticizers and super-plasticizers, retarders, stabilizers, water reducers, and anti-washout agents.

Aggregate: Loose, particulate materials, such as sand, gravel, pebbles, or crushed rock, added to a grout.

Batch: The amount of grout mixed at one time.

Bentonite: Clay mineral, preferably natural sodium montmorillonite. It is used to provide stability to a cement-based grout.

Blaine: The specific surface area of a particulate material, measured in sq. feet/lb. (cm^2/g or m^2/kg in SI units). Portland cements have a Blaine value of 1,460 – 2,440 sq. feet/lb. Ultra fine cements have a Blaine value of over 3,900 sq. feet/lb. and can reach 5,370 sq. feet/lb.

Bleeding: Water naturally separating from a cement-based grout at rest (decantation).

Bonding: Adhesion or the grip of cement to applied surfaces, i.e., interface strength.

Bulk density: The weight per unit volume of a material in its natural state.

Cement-based grout: A suspension mix of cement, water, and various admixtures and additives.

Chemical grout: A material generally comprising a pure solution, or, in the case of sodium silicates, a natural colloidal solution. Distinct from Particulate Grout (below).

Colloidal: A state of suspension in a liquid medium in which extremely small particles ($4 \times 10^{-8} - 4 \times 10^{-6}$ inches) are suspended, but not dissolved.

Colloidal mixer: A high-speed, high-shear grout mixer that produces a uniform, well hydrated particulate suspension.

Compaction grouting: Grouting using low mobility and high internal friction, grout (LMG) injected with less than 1 inch slump. Normally a soil-cement with sufficient silt sized component to provide plasticity, together with sufficient sand sized component to develop internal friction. The grout does not enter soil pores but remains in a homogeneous mass that gives controlled displacement to compact loose soils, and/or gives controlled displacement for lifting structures, and/or provides a controlled filling of large voids. Higher slumps may be used in void filling operations.

Compensation grouting: The injection of grout concurrent with underground tunneling to replace lost ground and prevent settlement of structures or the ground at the surface above the tunnel during construction.

Darcy's law: The velocity of flow of a liquid through a porous medium because of a difference in pressure is proportional to the pressure gradient in the direction of flow:

$$V = \frac{Q}{A} = K \times \frac{\partial h}{\partial L} \qquad \text{[Eq. 8-1]}$$

where,

V	=	velocity (feet/second)
Q	=	flow rate (cubic feet/s)
A	=	cross-sectional area (sq. feet)
K	=	coefficient of permeability (feet/second)
$\frac{\partial h}{\partial L}$	=	hydraulic (or pressure) gradient (dimensionless)

Emulsion: Colloidal particles dispersed and suspended in a fluid.

Fly ash: The finely divided residue resulting from the combustion of ground or powdered coal, which is transported from the fire box through the boiler by flue gases. Two types are typically used: C and F.

Fracture grouting: The injection of grout to intentionally fracture the ground hydraulically to create lenses of grout that strengthen ground by reinforcement action and/or produce controlled heave to lift structures.

Gel: A semi-rigid colloidal dispersion of a solid in a fluid.

Gel time: The time required for a liquid material to form a gel under specified conditions of temperature.

Grout: A cementitious material, chemical solution, or resinous material injected into a soil or rock formation to change the physical characteristics of the formation's material or mass after it has set or stiffened.

Groutability ratio of granular formations: The ratio of 15% size of the formation particles to be grouted to the 85% size of the grout particles (particulate grout).

Grouting: The injection under pressure of a fluid into the ground that then solidifies to alter formation's material or mass properties and/or create a structure.

Hydration: The process of a cement or pozzolan reacting chemically with water.

Laminar flow: Fluid moving in layers with a difference in speed between the layers (center layers moving more quickly).

Low mobility grout (LMG): Low slump grout, such as compaction-type grout, that does not travel freely and that becomes immobile when injection pressure ceases.

LMG grouting: The injection of stiff grout that displaces the soil into which it is injected, does not mix with or permeate the soil, and does not travel far from the point of injection. Also called Limited Mobility Grouting.

Lugeon: A measure of the permeability of a geological formation. One Lugeon unit = 1 L (of water)/meter of test hole/minute at an injection pressure of 10 bars (approximately 150 psi). The most common unit in which permeability is calculated by means of packer tests in conjunction with design or construction of grout curtain.

Mortar: A cement-grout, of low water/cement ratio, mixed with sand.

Newtonian fluid: In rheology, a fluid deforming for any applied stress.

OPC: Ordinary Portland cement.

PFA: Pulverized fuel ash or pulverized fly ash.

Particulate grout: Any grout characterized by undissolved (insoluble) particles suspended in the mix.

Percent fines: Amount, expressed as a percentage by weight, or a material in aggregate finer than a given sieve, usually the #200 sieve.

Permeability: A property of a porous solid that is an index of the rate at which a liquid can flow through the pores.

Permeation grouting: Filling of voids in a soil or rock mass with a grout fluid at a low injection pressure to strengthen and/or reduce permeability, while not destroying the original structure of the soil or rock.

Phreatic zone: The subsurface zone beneath the water table.

Portland cement: A cementitious material conforming to ASTM C150 with relatively high strength and slow and even setting.

Pozzolan: A siliceous or siliceous-and-aluminous material that possesses little or no cementitious value. In a finely divided form and in the presence of moisture, however, pozzolan reacts chemically with calcium hydroxide to form compounds possessing cementitious properties.

Pumpability: A measure of the properties of a particular grout mix to be pumped, as controlled by the equipment being used, the formation being injected, and the engineering objectives.

Resin: Any polymer, either natural or synthetic, that is a basic material for coatings and plastics. Used in grouting applications as the bonding material between rock bolts and rock.

Rheology: The study of deformation of viscous systems. Commonly used to refer to the collective fluid properties of grouts.

Set time (initial/final): Initial: when cement paste starts to harden and loses its plasticity. Final: when cement paste has hardened and lost all plasticity.

Slabjacking: The injection of grout beneath slabs or shallow foundations with the intent of producing controlled lifting.

Slurry grout: A fluid mixture comprising solids, such as cement, sand, or clays suspended in water (old term).

Soilcrete: An engineered mixture of cementitious materials with existing soils, for example as created by the jet grouting process.

Solution: A homogeneous molecular mixture of two or more pure substances. A true solution consists of particles less than 4×10^{-8} inches suspended in a fluid.

Stability (pressure filtration): A measure of the internal stability of a particulate grout when subjected to excess pressure in its fluid state. The higher the amount of water expressed during a standard test, the less stable the grout, and the less attractive it is for injecting into fissures and pore spaces.

Standpipe: Grout pipe projecting outside the rock surface and firmly bonded to the hole.

Thixotropy: The characteristic of increasing viscosity of the grout without agitation.

Tremie pipe: A pipe used to place grout underwater. The pipe is placed to the bottom of the hole. The end of the tremie pipe is always kept in the grout and never allowed to rise above the grout/water interface.

Turbo mixer: A mixer that circulates the grout mix components at high speed, without mechanical shearing.

Ultra-fine cement: A mixture of finely ground Portland or slag-based cement, often with mineral admixtures. Also known as microfine cement.

Unconfined compressive strength (UCS): The crushing force per unit area of a specimen tested without lateral confined support.

Viscosity: The resistance of fluid to flow, typically measured in cP (centiPoise).

Void ratio: The ratio of the volume of voids divided by the volume of solids in a given volume of soil or rock.

Water/cement ratio (w/c ratio): The proportion of water to cement historically measured by volume in the United States, but increasingly measured by weight. Ratio by volume = 1.5 × ratio by weight.

Water table: The upper surface of the groundwater profile in soil or rock, in the absence of overlying impermeable strata.

2.0 SOIL AND ROCK GROUTING

This section addresses soil grouting applications and technologies in Sections 2.1 through 2.7 and rock grouting in Section 2.8. Technologies for grouting of soils and rock formations have separately evolved through several years of application to solve a variety of geotechnical problems. Some techniques like compaction grouting, jet grouting and permeation grouting are well established. Others like soil-fracture grouting are relatively new in the United States. The grouting methods and materials used largely depend on the application and the in-situ geomaterial. However, it may be observed that for economic reasons alone, various fillers such as fly ash, sand, and gravel are usually incorporated into void filling grouts (Section 2.5), while other materials such as hot bitumen (Bruce et al. 2001, Bruce and Chuaqui 2012) are necessary to stop high flow/high head flows into quarries, and/or under dams.

2.1 Soil Grouting Applications

Soil grouting programs are used to achieve a variety of ground treatment objectives, and a number of soil grouting techniques are available. Soil grouting can be conveniently divided into two major groups of applications:

- Grouting for water control and waterproofing
- Structural grouting

Within each class of treatment, one or more of the grouting techniques may be applicable.

For the purposes of this chapter, waterproofing is construed to be used in conjunction with new construction, and water control to be used in conjunction with remedial applications. Techniques applicable for structural strength improvement are permeation, jet, soil fracture, and lime injection grouting, while LMG grouting can also be used for structural grouting, water control, and waterproofing.

The major structural applications of soil grouting are summarized below.

2.1.1 Densification

The density of all granular soils above and below the ground water table can be improved by various in situ techniques, such as dynamic compaction, vibro-compaction, stone columns, and compaction grouting. These are only applicable to new construction. For densification of loose granular soils under existing structures, compaction grouting has proven to be effective.

2.1.2 Raising Settled Structures

Successful raising of settled structures requires a controlled grouting operation. Although HMG grouting has successfully raised slabs and footings, its major disadvantage is the lack of control of the fluid mixes. Both compaction grouting and soil fracture grouting can be precisely controlled for structural settlement remediation or compensation.

2.1.3 Settlement Control

Depending on the soil type, cost, and potential cause of settlement, permeation, compaction, jet, and fracture grouting can be effective in controlling post-construction settlement.

2.1.4 Underpinning

A structure is normally underpinned to prevent settlement from occurring due to adjacent planned construction, or when it is proposed to add additional loads to a foundation. Depending on the soil beneath the structure to be underpinned, permeation, compaction, jet, and soil fracture grouting can offer alternatives to other underpinning techniques.

2.1.5 Excavation Support

Soldier piles and lagging, sheet piles, and structural diaphragm walls, with or without tiebacks or internal bracing, are the conventional methods of excavation support. However, when structures or utilities can be affected by the installation of these systems, permeation or jet grouting can be viable alternatives.

2.1.6 Soft-Ground Tunneling

Potential settlement is a design consideration on all soft-ground tunneling projects. Permeation, compaction, jet, and soil fracture grouting can be effective in preventing or compensating for this type of settlement.

2.1.7 Liquefaction Mitigation

Where structures are built on soils that are determined to be liquefiable, permeation, compaction, and jet grouting are potential methods for mitigating soils that are susceptible to liquefaction.

2.1.8 Water Control

Permeation and jet grouting have proven to be effective in controlling groundwater infiltration in underground construction elements, while existing structures experiencing

water infiltration can often be remediated by permeation, jet grouting, and the use of low mobility grouts.

2.2 Soil Grouting Advantages and Potential Disadvantages

2.2.1 Advantages

Soil grouting is an in situ treatment and so can usually offer a distinct economic advantage over removal and replacement. Another advantage over removal and replacement techniques is safety. For example, grouting for underpinning requires no excavation beneath structures, and thus eliminates the need for personnel to work in high-risk areas. Grouting is also generally less disruptive to the surroundings of the work site, and this can be of particular importance in residential areas. More sophisticated grouting technologies like compensation grouting can be used to achieve structural support during tunneling without impeding traffic flow on existing facilities.

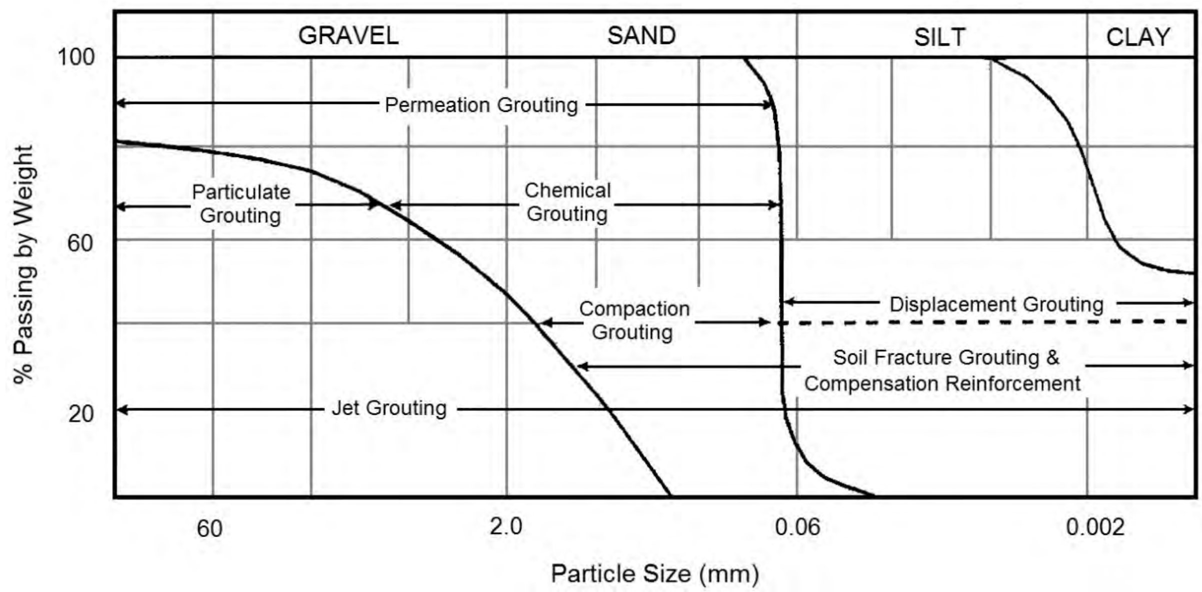
When using compaction grouting in finer, saturated soils, the instantaneous pressure exerted can fail to immediately squeeze the pore water pressures out of the fine-grained soils, so that densification or consolidation may not be achieved and simple displacement of the soil may occur.

Permeation grouting using certain chemical grouts may represent toxicity dangers to groundwater and the underground environment. Low toxicity chemical grouts, however are now sufficiently available for most purposes and should be specified except for unusual circumstances.

Jet grouting has the following advantages: has nearly unlimited configurations of column geometry, can be installed in areas of limited headroom, can be used in a wide range of soil types and groundwater conditions, and minimizes settlement.

2.2.2 Potential Disadvantages

The selection of the appropriate grouting technology is highly dependent on the soil type to be treated. Although the range of soil grouting techniques available encompasses most soil types, individual techniques are limited to specific soils, except for jet grouting, as shown in Figure 8-4.



Units: 1 mm = 0.04 inches

Figure 8-4. Range of applicability of soil grouting techniques.

In addition, the full scope and cost of the required program can seldom be determined accurately during the evaluation or design phase. Further, the effectiveness of some applications cannot be predicted with a great degree of certainty during the design phase. Each grouting method (especially jet grouting) can cause ground movement and structural distress. This must be carefully guarded against. Another limitation is the low level of knowledge on all aspects of grouting by the non-specialist engineering community and hence, an important objective of this chapter.

The disadvantages of jet grouting are: installation may cause ground heave, requires complex equipment, generated spoils must be disposed of or used as fill, and can be more difficult in plastic soils.

- Installation may cause ground heave
- Complex equipment
- Generated spoils must be disposed of or used as fill
- Can be more difficult in plastic soils

Other advantages and disadvantages of different jet grouting systems are shown in Table 8-1. It is to be noted that all systems have problems in very loose soils where the cement travels long distances, especially below the water table.

Table 8-1. Advantages and Disadvantages of Jet Grouting Systems

System*	Advantages	Disadvantages
Single Fluid	• Simple system for equipment and tooling • Good for sealing vertical joints • Good in cohesionless soils	• Smallest geometry created • Hardest to control heave • Difficult to control quality in cohesive soils
Double Fluid	• Most utilized system • Availability of equipment and tools • High energy, good geometry achieved • Most experience • Often most economical	• Very difficult to control heave in cohesive soils • Spoil handling can be difficult depending on fluid flows • Not usually considered for underpinning
Triple Fluid	• Most controllable system • Highest final quality in difficult soils (peat, soft clay) • Best underpinning system • Easiest to control spoil and heave	• Most complex system due to equipment and tooling • Requires significant experience
SuperJet/ MegaJet/ UltraJet/ STRAJet	• Lowest cost per volume treated • Best mixing achieved • Largest column diameters	• Requires special equipment and tooling • Difficult to control heave in cohesive soils • Spoil handling may be difficult due to high flows used • Cannot work near surface without support • Highest logistical problems
X-Jet	• Confidence of geometry • Controllable material costs • Best for soft, cohesive soils	• Very specialized equipment that requires daily calibration • Limited experience available

* See Section 2.7 of this chapter for additional details of each jet grouting system.

2.3 Permeation Grouting

Permeation grouting is defined as the introduction of low viscosity solutions such as particulate suspensions or chemical grouts into the ground, e.g., clean sands and gravels or permeable discontinuities in rock without disturbing the structure of the ground (Littlejohn 2003). Permeation grouting is utilized to reduce permeability or increase strength of the soil, or make the structure or volume of the original soil mass more homogeneous and cohesive.

The type of grout utilized depends on the grain size of the in-situ soil and the desired results from the grouting operation, as previously illustrated in Figure 8-4.

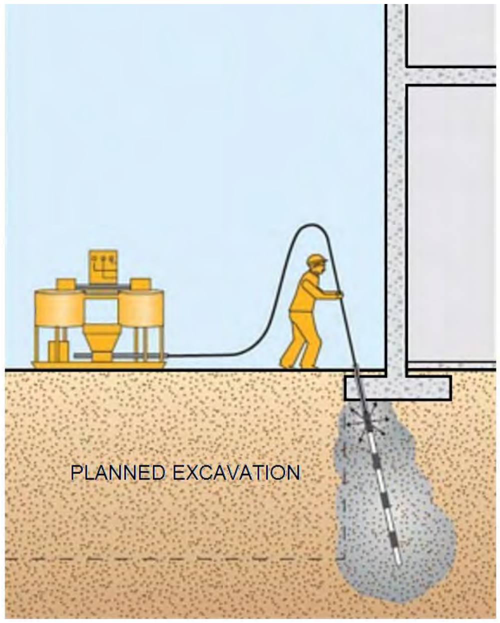
The term “structural permeation grouting” is applied where the objective of the grouting is to improve the strength and/or rigidity of the groutable soils to prevent ground collapse, reduce otherwise unacceptable ground movement during construction, improve bearing capacity, etc. The term “waterproof grouting” is used to describe permeation grouting aimed primarily at stopping the flow of water, which otherwise would provoke ground movements or the flow of unacceptably large amounts of water into a construction area, or both. Underpinning is another application of structural grouting, wherein granular foundation support soils are strengthened so as to permit excavation adjacent to footings.

Permeation grouting is intended to fill all (or most, i.e., 70% to 80% of) the natural pore spaces in a soil mass, without changing the virgin structure or volume. Grouts can thus be used to increase the cohesion between soil particles, thereby leading to increased strength parameters and/or reduced permeability. As a general rule, the finer the pores, the higher the cost of the grout; therefore, it is normal to attempt to fill larger pores first with conventional particulate grouts, and to permeate into finer or residual pores with chemical grouts, or ultra-fine grouts.

2.3.1 Applications

Permeation grouting is used to improve the characteristics of soils, and can be used for the following applications:

- Waterproofing, typically for remedial purposes, such as subway tunnels, sealing off water ingress in mining (Littlejohn 2003) and vertical diaphragm walls (Town 2012)
- Seepage control
- Slope stabilization
- Soil strengthening to reduce lateral support requirement (Mitchell 1981)
- Settlement control, underpinning and excavation support of granular soils during excavation
- Soft ground tunneling to increase cohesion, as shown in Figure 8-5.
- Mitigating the need for liquefaction retrofit by increasing density and displacing pore water



Courtesy of Hayward Baker, modified from http://www.haywardbaker.com/

Figure 8-5. Tunnel excavation support using permeation grouting.

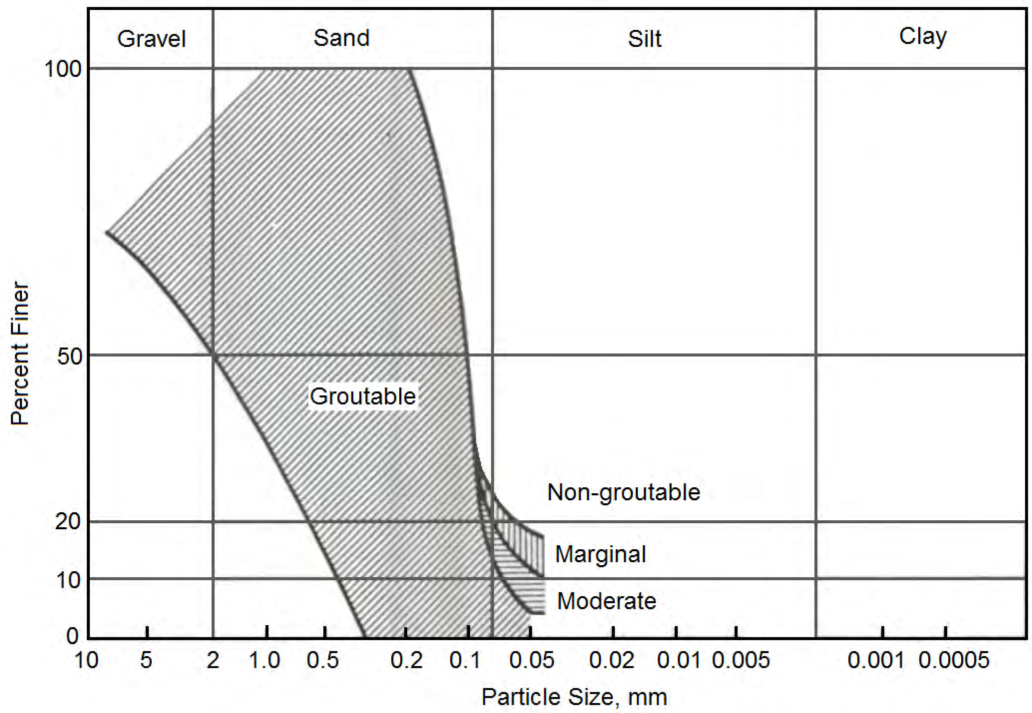
2.3.2 Feasibility Evaluations

The feasibility of using permeation grouting depends on factors such as soil type, stratigraphy, site history, permeability of the soil, grout properties and its effects on groundwater. These factors are discussed in the following subsections. A "groutable" soil is one that will, under practical pressure limitations, accept permeation by a given grout at a sufficient flow rate to make the project economically feasible.

2.3.2.1 Geotechnical

Particulate grouts can be used for permeation grouting only in coarse soils, such as medium to coarse sands and gravels. The permeability of sands may vary as much as three or four orders of magnitude, from 0.4 inches/second for medium-grained clean sands to as low as 4×10^{-5} inches/second for sand containing 25% or more silts and clays. For very low permeability sands, the injection rate at permissible pressures may be so slow that grouting becomes unfeasible. *Thus, permeation grouting is recommended only in predominantly sandy materials with less than 15% silts and clays.*

Soils are initially classified as *readily groutable* if they have less than 12% fines, *moderately groutable* for 12 – 15% fines, and only *marginally groutable* for 15 – 20% fines. Sands are usually considered ungroutable if they have more than about 20% fines. Figure 8-6 shows typical grain-size ranges for soils amenable to permeation by typical silicate grouts.



Units: 1 mm = 0.04 inches

Karol 2003

Figure 8-6. Typical grain size curves for permeating soils.

Groutability ratios were studied by King and Bush (1963) to determine the applicability of permeation grouting in soils and rocks. The ratios, which show the relationship between grain size of suspended materials in cement based grouts and pore size in granular soils were supported by the work of De Paoli et al. (1992), who confirmed that the limits of penetrability could be enhanced by using correctly balanced (i.e. stable, low cohesion) grouts (DePaoli et al. 1992).

Groutability ratios, N, and N_C for soils:

$$N = \frac{(D_{15})_{SOIL}}{(D_{85})_{GROUT}} \qquad \text{[Eq. 8-2]}$$

$$N_C = \frac{(D_{10})_{SOIL}}{(D_{95})_{GROUT}} \qquad \text{[Eq. 8-3]}$$

Grouting is consistently possible in soils if $N > 24$ or $N_C > 11$, and is not possible when $N < 11$ or $N_C < 6$. Suitability of materials for grouting also depends on the soil particle size. Applicability of soil sizes for specific cements and bentonite are as follows:

- Type I and Type II Portland cement: Soils coarser than 0.024 inches
- Type III Portland cement: Soils coarser than 0.016 inches
- Bentonite: Soils coarser than 0.01 inches
- Microfine cement: Soils coarser than 0.002 inches

Microfine cements have also been used for fracture grouting as opposed to permeation grouting in very fine soils such as silts and clays. No distinct relationship was observed for ultrafine cement grouts between penetrability and maximum grain size of the cement particles (Warner 2003). Penetrability of such grouts depends on grain shape, surface condition and chemistry (polar strength) of cement particles. Slag-based cement grouts are weakly polar whereas ultrafine cement grouts exhibit strong polarity, thereby requiring high shear mixing of the grout to improve penetrability for the latter.

Injectability of soils can be approximated using Hazen's equation, which provides an estimate of hydraulic conductivity of the soil and was found to be fairly accurate for undisturbed sandy soils (Lees and Chuaqui 2003):

$$k\ (\text{feet/seconds}) = C\,(d_{10})^2 \qquad \text{[Eq. 8-4]}$$

Landry et al. (2000) provided the following estimates for injectability, while recommending small-scale field testing to determine injectability for the final grouting work. The groutability of soils is measured in terms of their initial permeability (prior to grouting).

- Readily groutable: $0.04 \leq k \leq 4 \times 10^{-4}$ inches/second
- Injectable with regular cement grouts: $k > 0.04$ inches/second
- Marginally groutable: $4 \times 10^{-4} \leq k \leq 4 \times 10^{-5}$ inches/second
- Injectable with microfine cement grouts: $k > 0.002$ inches/second
- Practically ungroutable: $4 \times 10^{-5} \leq k \leq 4 \times 10^{-6}$ inches/second
- Injectable with solution grouts: $k > 4 \times 10^{-5}$ inches/second

Chemical grouts hold an advantage over particulate grouts in terms of ability to penetrate smaller pores, have lower viscosity, and can be better controlled over setting time. They are however more expensive and are a more complex technology. Permeation grouting using chemicals is most effective when the fines content of soils is less than 10 percent, less effective when fines are greater than 15 percent and is not possible when fines content is greater than 20 percent.

2.3.2.2 Environmental

Sodium-silicate and acrylamide are two grouts that were used in early grouting trials. However, incidents of water poisoning linked to acrylamide grouting in Japan in 1974 led to a subsequent ban on all chemical grouting compounds except non-toxic silicate-based grouts. Acrylate grouts, which are a more environmental-friendly replacement with properties similar to acrylamide grouts are used by a specialized sewer-sealing industry in the United States.

2.3.2.3 Project Conditions

Grout selection for permeation grouting is also affected by project conditions. Analysis of structural requirement and permeability of soil at the project location is an important part of permeation grouting design. Gradually thickening grout mixes containing an initial w/c ratio of 2.0 were used regularly in civil engineering practice after 1991 (Littlejohn 2003). However, high pressure injections with dilute, unstable cement grouts having w/c of 4.0 to 8.0 and high velocities were required to prevent early sedimentation and enable longer penetration distances in the gold mines of South Africa, where high temperatures led to formation of grout barriers at depths greater than 3,000 feet.

2.3.3 Design Considerations

The early establishment of clear, quantitative objectives to be achieved by a permeation grouting program is a basic prerequisite to good design and satisfactory, economical performance. A successful grouting program requires the selection of a suitable grout material, the correct drilling equipment, procedures, and grout hole pattern. The design objective for structural grouting is often to give non-cohesive ground (no strength under unconfined conditions) sufficient cohesion to prevent the beginning of collapses or soil “runs” into excavations, tunnels, or shafts.

Although many grouts (including properly formulated particulate grouts) can be considered to be permanent, i.e., have a service life in excess of 20 years under normal conditions, most structural chemical grouting is required for only a few days to several months. Sodium silicate grouts cannot be regarded as permanent (Naudts 1995, Baker 1982). In the case of grout underpinning, the soil strength lost by the reduction in confining stresses is replaced by the cohesion imparted to the soil by the grout and hence results in a permanent effect.

The spacing of grout holes for permeation can be accurately designed using well-defined equations (Xanthakos et al. 1994). These require knowledge of the granulometry of the soil and the rheology of the grout, as well as the anticipated flow rates and limiting pressures.

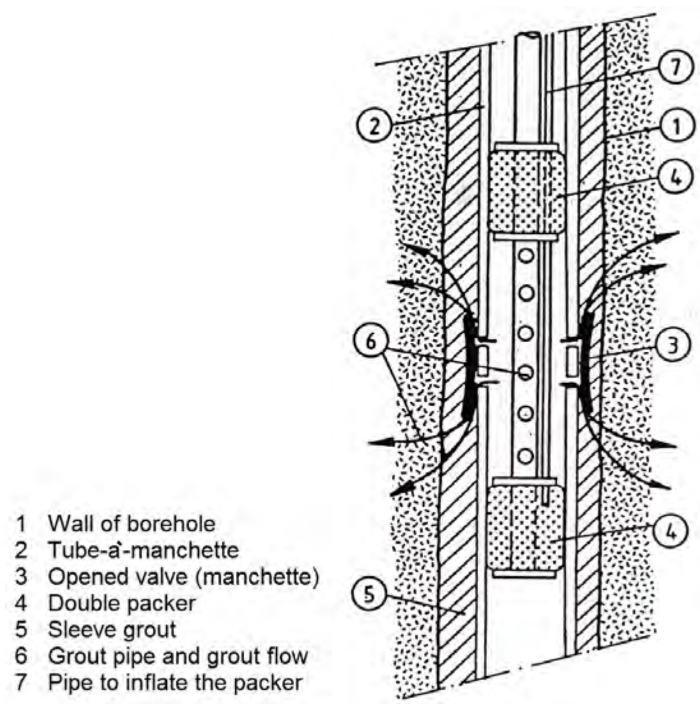
However, for preliminary cost and feasibility evaluations, the guidelines summarized below may be considered.

2.3.3.1 Spacing

Spacing of grout pipes may vary from 1.6 to 5 feet for waterproofing and from 3.3 to 5.2 feet for structural applications. A typical number for each is 4 feet.

2.3.3.2 Equipment

Sleeve-port grout pipes, originally called "tubes-à-manchette," shown in Figure 8-7, should be used on all permeation grouting projects, as opposed to basic, end-of-casing material injection as the casing is gradually withdrawn.



Kutzner 1996

Figure 8-7. Mode of operation of a tube-à-manchette.

Sleeve-port grout pipes allow for a well-planned primary-secondary grout program horizontally and vertically. The system consists of a one- to two-inch diameter plastic pipe that has grout holes drilled through the pipe wall at distinct vertical locations, usually 1-foot centers. The grout holes are covered with a rubber sleeve that acts as a one-way check valve. The grout pipe is installed in a slightly oversized borehole, and the annular space between the pipe and the borehole wall is filled with a brittle but weak cement-bentonite grout. This grout sheath is fractured when the sleeve is expanded by grouting pressure from inside the pipe, utilizing a double packer. The sleeve-port can be injected in any sequence (although always

from the bottom up) and may be re-injected, if desired. These ports can also be tested with water to roughly estimate the permeability of the soil before or after grouting. The grout pipes can also be used to run cross-hole shear wave velocity tests before and after grouting.

The permeability of the soil in both horizontal and vertical directions should be evaluated in order to predict the relative shape of grout bulbs. It is a common experience to observe elliptically shaped, isolated grout bulbs, with height to diameter aspect of about 0.80, because the horizontal permeability is greater than the vertical permeability. Soil anisotropy will affect the selection of grout pipe spacing and grout port spacing, as well as the sequence in which primary and secondary holes are grouted.

If unexpected, ungroutable lenses occur periodically throughout the design-grouting zone, they will control and greatly influence the direction and migration of grout from the grout pipe location. If major ungroutable pockets are encountered, their presence, especially if unanticipated, will significantly influence the effectiveness of the grouting program.

The original stratigraphic profile should be confirmed during the borings conducted for placement of grout pipes. Since wash borings and split spoon samples are generally obtained during grout pipe drilling and the drillers may not be experienced in geologic drilling, it is important that they report all observed changes in response to the drilling, including changes in drilling rates and wash water.

2.3.3.3 Grout Quantities

In order to calculate the volume of grout needed to treat a given soil volume, one must have a fairly accurate estimate of the porosity of the soils to be grouted. Typical groutable soils have porosities of 0.25 (finer grained) to 0.45 (coarser grained), and it is common to assume that the total void space will be filled with grout. For a porosity of 0.35, 92.5 gallons (350 liters) of grout will be required for every cubic meter (1 m^3 = 1.308 cubic yards) of soil treated. Depending on the grain size curve analysis, it may be possible to treat larger pores first, with an appropriate, economical, particulate grout. So, in this case, the 35% porosity may be split 10% particulate, and 25% chemical, for example. Because a major cost of permeation grouting is the chemical grouts, the porosity has important cost consequences. Estimates of soil porosity are often obtained from correlations with Standard Penetration Test “N” values. Where relatively undisturbed samples are obtained, unit weight and specific gravity test results can provide a better estimate of soil porosity for use in grout volume calculations. Equally, permeability tests conducted prior to grouting will give a good indication of the amenability of the soil to different types of grout. Depending on the scale of the project, it will be prudent to add an extra 5% to 15% grout volume to compensate for “edge dilution” effects.

2.3.3.4 Construction Equipment

All permeation grouting equipment for chemicals should be of a type, capacity, and mechanical capability suitable for doing the work. Equipment for use with particulate grouts is described in Section 4.

Pumps. The permeation grout plant is usually of the continuous mixing type and should be capable of supplying, proportioning, mixing, and pumping the grout with a set time between 5 to 50 minutes. Batch-type systems can also be used with appropriate QA procedures. Each main pump should be equipped with sensors to record pressure and rate of injection, as a minimum. The sensors should be constructed of materials that are non-corrodible for the intended products and should accurately operate, independent of the grout's viscosity. The pumping unit should be capable of varying the rate of pumping, while maintaining the component ratios constant.

Piping and Accessories. The pumping unit for permeation grouting should be equipped with piping and/or hoses of adequate capacity to carry the base grout and reactant solutions separately to the point of mixing. The hoses should come together in a "Y" fitting containing check valves to prevent backflow. The "Y" fitting should be followed by a suitable baffling chamber. A sampling valve should be placed beyond the point of mixing and the baffling chamber, and should be easily accessible for sampling mixed grout. A water-flushing connection or valve should be placed behind the "Y" to facilitate flushing the grout from the mixing hose and baffle between grouting sessions. Distribution of proportioned grout, under pressure, to the grouting locations should be monitored by separate, automatic recording, flow rate indicators, and gages. Batch mixing does not require such "Y" fittings, as the reacting grout is pumped to the hole through one line.

Chemical Tanks for Chemical Grouting. Chemicals should be stored in metal tanks, suitably protected from accidental discharge through valves and other necessary means. Tank capacity should be sufficient to supply at least one day's worth of grouting materials so as not to interrupt the work in the event of chemical delivery delays.

2.3.3.5 Grout Types and Selection

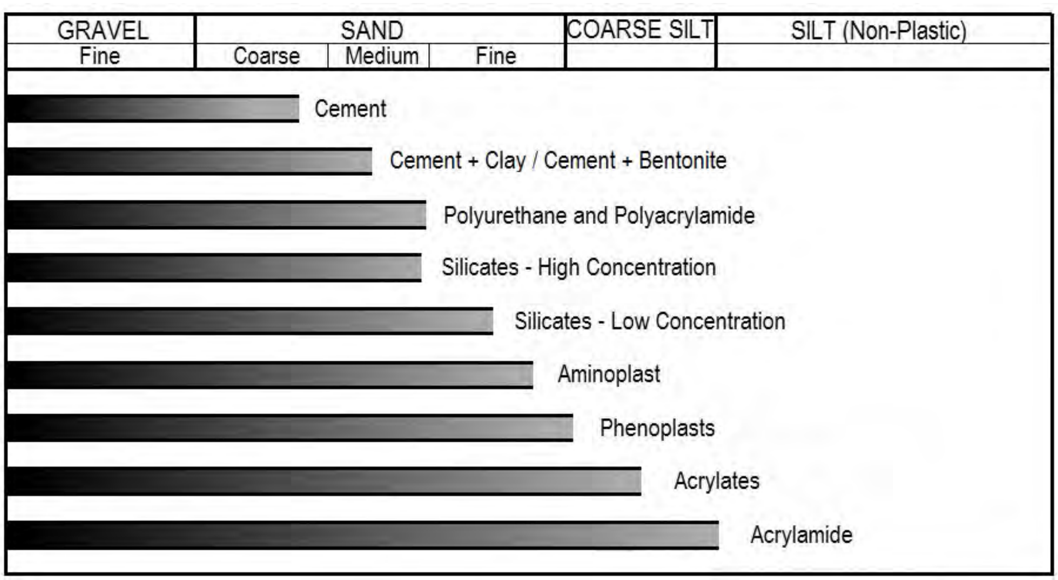
Two types of grouts are used for permeation grouting: a) chemical grouts, which consist of various materials in solution, and b) particulate grouts, which consist of cement, soil, clay or a mixture of these materials. Research activities have shown economic, technical, and environmental benefits in favor of cement-based grouts (Weaver and Bruce 2007, Xanthakos et al. 1994). Researchers have also shown that cement particle size, viscosity, and internal stability (pressure filtration coefficient) control the effectiveness of a cement-based grout

more than its ability to penetrate fine-grained soils. Selection of the grout material should take into consideration water flow rates, gel times, and durability. The required residual permeability of the grouted mass will also affect grout selection.

Typical water-cement (w/c) ratios in cement-based (water and cement) grouts should vary from 0.5 to 6. Lower w/c ratios result in higher strength, less segregation and filtering but are harder to inject than those with higher water content (Mitchell 1981). Increased permeation, prevention of cement flocculation and control over setting time may be achieved by adding chemical additives. The ratio of soil to cement by volume in soil-cement grouts typically varies from four to six, with w/c ratios varying from 0.33 to 2.

The most commonly used compounds for producing chemical grouts are silicates, lignin, resins, acrylamides, and urethanes. Silicate grouts, primarily sodium silicate, are extensively used for permeation grouting whereas others materials find limited use due to higher cost and toxicity. Grouts having 25 to 30 percent silicate content are used for waterproofing applications, and higher concentrations of 40 to 60 percent are used for improving structural strength.

Conventional Portland cement can only permeate into gravels and coarse sands due to its larger particle size. Fine-grained cements (e.g., ultra-fine cement) were introduced into the United States in 1983, and can be used to prepare grouts capable of permeating finer sands without forming a filter cake at the borehole. The exact choice of grout type depends largely on the grain size distribution and hence, permeability of the soil mass. The penetrability of various grouts is shown in Figure 8-8.



After Karol 1990

Figure 8-8. Penetrability of various grout types.

2.3.4 Cost

To determine a preliminary estimate for permeation grouting quantities, the volume of ground to be treated (cubic yards) is multiplied by a projected 30% grout volume factor. Next, this volume is converted to quarts by multiplying by 808. For a project where more than 200,000 quarts of sodium silicate grout are anticipated, a cost of $0.65 per quart ($2.50/gallon) in-place can be used for estimating purposes.

A mobilization/demobilization rate ranging from $10,000 to $50,000 and a cost of providing and installing the sleeve port grout pipes starting at a minimum of $20 per linear foot should be added to the estimate. This preliminary cost estimate would be applicable for any particulate permeation grouting.

2.3.5 Case Histories

2.3.5.1 Case History 1: Multiple Pass Permeation Grouting to Encapsulate and Contain Radioactive Waste at Oakridge National Laboratory, Oakridge, Tennessee (Naudts et al. 2012)

The grouting program developed for this project sought to hydraulically isolate 34 million quarts of Liquid Low-Level Radioactive Waste (LLLW) contained in two trenches at the Oakridge National Laboratory. A multiple-pass, multiple-stage, multiple-hole grouting program was conducted inside the trenches and grout curtain was later constructed in the soil around and below the trenches. Permeation grouting was performed via vertically driven steel-sleeve pipes using five different types of stable, balanced, durable cement-based suspension grout mixes with various rheologies. The grout curtain was constructed using acrylamide grout due to its ability to permeate fine fissures and fractures. The grouting operations were monitored using real-time data collected using the Computer Aided Grout Evaluation System (CAGES) and the final permeability around the trenches was reduced to values lower than the target permeability of 4.0×10^{-6} inches/second.

The grouting program was designed to encapsulate and contain the radioactive waste located in Trench 5 (295 feet long by 15.7 feet deep) and Trench 7 (three separate cells, 98 feet long by 15.7 feet deep) for a target minimum period of 200 years. A small-scale construction verification trench with the same geology as the original trench was grouted to confirm that design objectives can be met and to make necessary design adjustments. The challenge faced was that the gravel had to be 100% grouted without allowing water or grout to surface, and completion could only be monitored by the CAGES monitoring system.

Laboratory tests were conducted to develop cement-based suspension grout formulations with specific rheological characteristics to facilitate multiple passes. Class F fly ash was

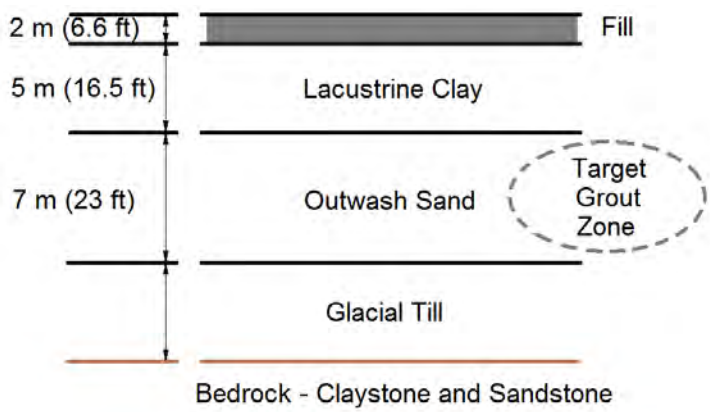
added to achieve the required durability, slow down hydration, and reduce thermal shrinkage. Pre-hydrated biopolymer solutions were used to reduce pressure filtration coefficient and prevent 'dry-packing' of the grout. Superplasticizer was used to lower the viscosity and cohesion of the grout. The final composition of acrylamide grout, which was prepared as a two-component system consisted of water, 40% acrylamide solution, triethanolamine solution, ammonium persulfate, 1% potassium ferricyanide (KFe) solution, sodium bicarbonate (baking soda), and dye (blue and red).

The Lugeon values of the soils eventually dropped to zero after 4 to 5 grouting passes spread over several days. A total of 9,000 cubic feet of grout was installed in Trench 5, 5,332 cubic feet of grout was installed in Trench 7, and 12,570 cubic feet of acrylamide grout was installed in both trenches combined. The in-situ hydraulic conductivities (K) measured post-grouting showed a decrease of up to five orders of magnitude for the crushed stone within trenches (4×10^{-3} to 4×10^{-8} inches/second), while that of the soil surrounding the trenches decreased by two orders (from 4×10^{-5} to 4×10^{-7} inches/second).

The project showed that low-pressure, permeation grouting can be used to safely and effectively control liquid radioactive wastes disposed in burial trenches. The grouting program was professionally executed to obtain a soil mass with very low residual permeability without drilling and without bringing contaminants to the soil surface.

2.3.5.2 Case History 2: Permeation Grouting in Outwash Sands, Edmonton, Alberta, Canada (Brachman et al. 2004)

This case history describes the field trials conducted in Alberta, Canada to evaluate three different types of permeation grout – sodium silicate, microfine powder, and microfine cement. The purpose of conducting field trials was to study whether permeation grouting is suitable for providing temporary support to the sandy soil during tunnel construction. Soil conditions were similar to those expected for the proposed City of Edmonton light rail transit extension, which involves construction of two tunnels, each with a diameter of 20.33 feet and length of 1,155 feet, passing through a sandy outwash deposit. The soil at the field test location consisted of five layers as shown in Figure 8-9.



Brachman et al. 2004

Figure 8-9. Soil conditions at field trial location in Edmonton, Alberta.

Grain size analysis was performed on samples taken from grout holes drilled in the sand to determine percent fines (material finer than No. 200 sieve. Average fines content varied from 1% to 28%, which showed that the poorly-graded sand was mostly readily groutable (11 of 13 samples), one sample being moderately groutable and one sample being not groutable based on groutability ratio (defined in Section 2.3.2.1).

The grouts were injected through 12 grout holes. The injection points were spaced at 4 feet center-to-center along a triangular grid at depths ranging from 34.5 feet to 45 feet. The composition of each grout was designed as shown in Table 8-2.

Table 8-2. Components of Permeation Grouts Used by Brachman et al. 2004

Component	Grout A	Grout B	Grout C
Water	25 quarts (Liters)	86 quarts (Liters)	51 quarts (Liters)
8% Bentonite slurry	3.5 quarts (Liters)	14 quarts (Liters)	
Grouting material*	44 lbs. (20 kg)	132 lbs. (60 kg)	
Rheobild 1000	1.2 quarts (Liters)	1 quart (Liter)	
Delvo	0.8 quarts (Liters)		
Welan gum	0.044 lbs. (20 gm.)	0.077 lbs. (35 gm.)	
Sodium silicate			45 quarts (Liters)
Hallco C-491 (Neutralizing agent)			4 quarts (Liters)
Grout hole positions	1 – 3	4 – 7	8 – 12

Seismic velocities were recorded using seismic velocity holes for all three grouts after four days. The increase in wave velocity was correlated to the increase in strength of materials between the source and receiver. It was observed that the microfine cement, which required very high injection pressures during grouting did not produce a uniform grouted mass. Microfine powder did not cause an increase in strength of the sand even after one month. This failure was attributed to low temperatures, lack of oxygen. and poor mixing of components in the ground, which inhibited curing of the grout. Sodium silicate was the most successful grouting material, resulting in a well-permeated hard soil mass.

2.4 Compaction Grouting / LMG Grouting

Low mobility grout (LMG) grouting is defined as “the injection of a stiff grout that does not mix with or penetrate the soil, often displaces the substrate into which it is injected, and does not travel very far from the point of injection” (Byle 1997). LMG grouting is also known as limited mobility grouting. It consists of the injection of low slump, low mobility grout (LMG) into loose or loosened soils of appropriate grain size distribution. Alternatively, similar LMGs can be injected into voids in rock masses as a bulk infill material or as a component in a seepage remediation grouting program. Bulk infill grouting is presented in Section 2.5.

2.4.1 Applications

LMG is used in the following applications:

- Compaction grouting
- Sealing of flowing channels – Injection of LMG with water reducing and viscosity modifying admixtures to provide cohesiveness, prevent washout, and effectively seal off flow in subsurface conduits
- Pre-grouting of large fractures – Injection of LMG into large fractures to reduce opening sizes to make high mobility rock grouting (fissure grouting) more effective
- Abandoned mine filling – Prevention or remediation of mine collapse
- Structural supports – Injection of LMG to create grout columns to act as structural support (underpinning) for buildings and other structures
- Grout jacking – Injection of grout beneath slabs or structures that have undergone settlement, with the objective of lifting them back into position
- Soil reinforcement
- Post-grouting of deep foundations

Uses of compaction/LMG grouting include the following:

- Correction of differential settlements of structures/raising of surficial structures
- Soil densification (for static and seismic enhancement)
- Structural underpinning
- Ground strengthening adjacent to open excavation or tunneling
- Settlement control over tunnels or sinkholes, as shown in Figure 8-10
- Sealing off major water ingress through open channel systems

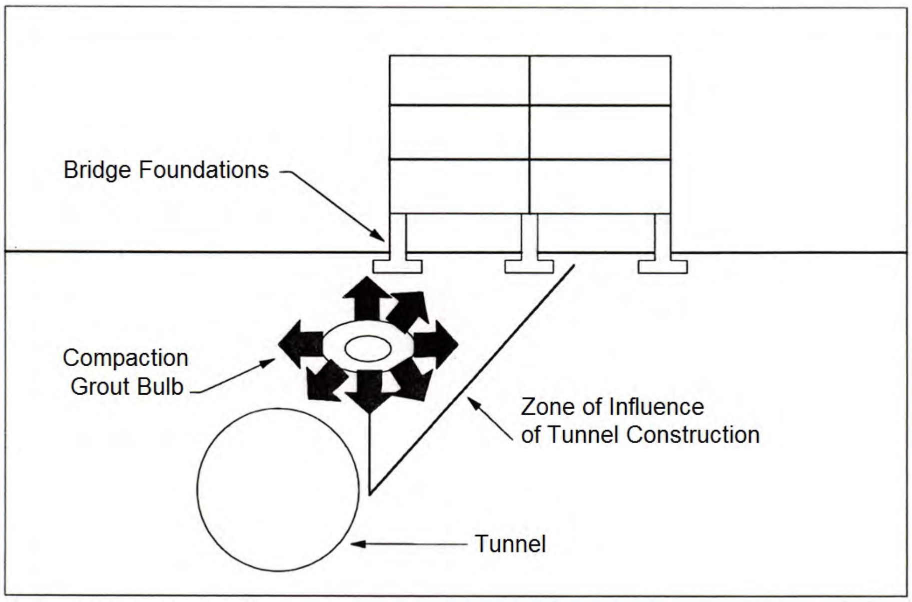


Figure 8-10. Prevention of tunnel-induced settlements using LMG grouting as compensation for loss of ground.

2.4.2 Feasibility Evaluations

As in all specialty geotechnical processes, the input of a specialty contractor should be sought beginning with the development of a well-conceived test program. This is especially valid when the purpose of the compaction grouting is to raise a settled structure or to compensate for ground loosening under the foundations of an existing structure adjacent to active soft ground tunneling. There are no mathematical equations to accurately design grout hole spacing, rates of injection, limiting volumes, and so on, as is the case with permeation grouting. There is, however, a great deal of project experience and a large number of successful case histories well documented to guide project implementation. A few representative case histories are presented in Section 2.4.5. The geotechnical, environmental,

and site condition factors which should be considered to assess compaction grouting feasibility are discussed below.

2.4.2.1 Geotechnical

Densification of soils by compaction grouting may be expected to be effective in all relatively free-draining soils, including gravels, sands, and coarser silts. In fine-grained soils, pore pressures may not be able to dissipate and improvement may not be economically achievable. Grout mix design is also critical in that the grout must have high internal friction to ensure that the bulbs preserves their "spheroidal" shape in the soil. Otherwise, fracturing and lensing will occur, leading to ineffective densification (Warner et al. 1992). For LMGs used to fill voids or for water cut off, different rheological properties may be preferable, e.g., a slightly higher acceptable slump (up to 3 inches) or the use of polypropylene fibers in the mix.

2.4.2.2 Environmental Conditions

Compaction grouting has minimal adverse impact on surface environment and soils due to its confinement to the grout's zone of influence. The use of digital computer controlled directional drilling with grouting offers very high levels of control to permit safer and more precise injection. Use of advanced injection control and imaging techniques improves the understanding of soil-grout interaction under varying conditions.

2.4.2.3 Project Conditions

Regarding site assessment, conventional measurements, such as SPT, CPT, are typically used. For sinkhole remediation or flow sealing, piezometric data and a variety of geophysical techniques (e.g., ground penetrating radar (GPR), electrical resistivity, electromagnetic imaging and tomography) can provide more far-ranging data than the point-specific information from a single borehole. Site conditions would also always include access.

2.4.3 Design Considerations

2.4.3.1 Spacing

For compaction grouting for densification or re-densification, grout pipes are typically installed at 8 to 15 feet intervals for tunneling projects, 6.6 to 16 feet intervals for site improvement, and 3.3 to 10 feet for remedial work on existing structures. Primary holes for use in locating and sealing sinkholes or channel flows should be spaced in relationship to the nature of the problem, but are typically in the range of 10 to 30 feet. In such instances, tertiary holes are usually required to ensure and verify satisfactory performance. The grout

pipe diameter should be at least 3 inches to transmit the specified low slump material without plugging or to minimize shear resistance.

2.4.3.2 Grout Quantities

LMG quantities will depend on the soil type, its existing density, and the density required, or on the size of the void to be filled. For most densification projects, the volume of LMG will range between 3 to 12% of the volume of soil being treated, whereas for void filling, individual stages of grouting may consume tens of cubic meters of grout.

2.4.3.3 Construction Equipment

Mixers and Pumps. The low mobility grout will require different mixing, pumping, and delivery equipment than more fluid grouts. "Compaction Grouting - the First Thirty Years" lists the requirements for the mixers, pumps, and hoses (Warner 1982). Specialized contractors and some grout equipment suppliers have developed their own equipment and continue to update this equipment based on their own, on-the-job experiences and requirements. Pumps must be able to inject at rates from ½ cubic feet/minute upwards.

Obviously, the grout plant should be designed to handle the specified materials for this type of work. The mixer should be a pug mixer type to ensure complete uniform mixing of the materials used and should be of sufficient capacity to continuously provide the pumping unit with mixed grout at its normal pumping rate. The pumping unit should be capable of continuously delivering the specified grout materials at appropriate rates and pressures to the grout pipe head. Under certain conditions, it may be possible to use ready-mix material delivered in mixer trucks to the pumping location. Each truck's load must be carefully tested to ensure compliance with the slump criterion. The inspector should be prepared to reject truckloads that exceed criteria upon delivery, or at any time during the pumping operation from that batch.

In general, the contractor has more control over the properties and consistency of the grout when he batches it on site. In addition, site batching can limit material wastage and delays.

Grout Pipes. Grout pipes should be steel casing of adequate strength to maintain the hole and to withstand the required jacking and pumping pressures. It is usual to inject the grout while withdrawing the pipe from the maximum depth in well-defined steps ("stages"), ranging typically from 1 to 3 feet.

2.4.3.4 Grout Types and Selection

Soil-cement mixture grouts with lower w/c mixes are used for high viscosity LMG (Mitchell 1981). LMGs with zero slump are made using cement, clay, and fly ash mixes. Portland Type I or II cement is normally used. Fine aggregate is usually sand with a fines content of not less than 10% and not more than 25%. Natural fines may be supplemented with fly ash, bentonite, or aggregate washings. Proportions of the mixture are approximately three to six sacks of cement per bulk cubic yard of silty sand and water as required to achieve a pumpable mix with not more than a 1 in. slump as measured at the grout pipe header. Depending on the application, other additives may include gravel, coarse sand, fibers, or anti-washout agents. Similarly, in certain cases, e.g., sinkhole remediation in a dam core, no cement is added to assure that no "hard point" is created in the dam.

2.4.3.5 Grout Injection

The optimal rate of grout injection for compaction grouting usually falls within a range of 1.5 to 2.0 cubic feet per minute (Warner and Byle 2012). The rate of injection greatly impacts the effectiveness of grouting program. Excessive pumping rate, particularly in fine grained soils, can cause excess pore pressures to build up that can cause damage to adjacent structures and a higher rate of injection will produce a lower quantity of grout to be injected prior to refusal reducing the effective radius of improved soil. This will require more grout holes to achieve the same level of improvement. In sensitive areas such as near retaining walls, downslopes, or in water retaining embankments, slower rates should be used. For compaction grouting, jacking, and void filling, injection should be completed on perimeter holes prior to those on the interior; and injection from the perimeter inward is typical.

2.4.4 Cost

A split-spaced grid pattern is utilized, with the grid pattern spacing and the volume to be injected dependent upon the required increase in density in the formation or the size of the void to be filled. As a result, LMG grouting costs vary from as low as $5.00/cubic yard of soil treated to more than $50.00/cubic yard, plus mobilization and pipe installation costs. The cost variation in projects, drilling costs, mixtures, quantity injected, rate of injection, etc., makes this system particularly sensitive to price fluctuation. The cost of the grout alone is in the range of $60 – $120/cubic yard.

2.4.5 Case Histories

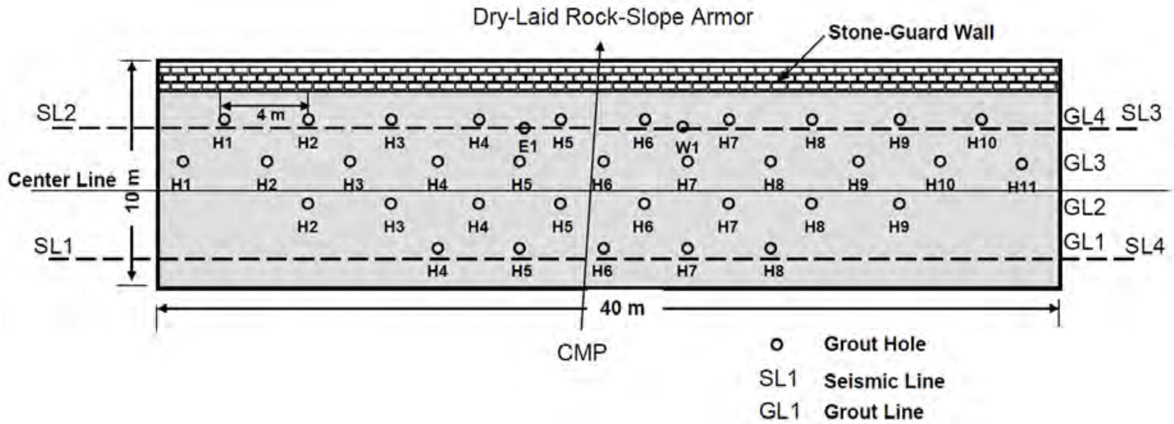
2.4.5.1 Case History 1: Rocky Mountain National Park, Colorado (Haramy et al. 2012)

Compaction grouting was used to stabilize roadway settlement caused by piping of a failed culvert in Rocky Mountain National Park, Colorado. Rehabilitation of the historic placed-rock armored slope structure, which is located below roadway grade along the outboard lane was performed without affecting its historic and aesthetic value.

The grouting program successfully used a volumetric, non-destructive test method based on seismic measurements to collect and analyze data describing subsurface conditions over the entire project area. The advantage of this NDT method is that the data represents the condition of the entire ground mass as opposed to the Standard Penetration test (SPT) or other conventional "point test" which cannot collect spatially and temporally continuous, subsurface condition data.

The site consisted of a 65 feet high and 131 feet long rock slope armor with a 2 feet culvert at the center, which resulted in settlement of the pavement measured at over 4 inches per year. A break in the culvert was observed from internal video inspection, which indicated piping caused by groundwater flow. Planned mitigation measures included drainage improvements, embankment stabilization and armor stabilization.

The embankment rock armor materials were stabilized by injecting grout into holes drilled in the pavement. The culvert was lined to prevent further piping and to keep the water away from the fill, the embankment rock armor materials were stabilized by injecting grout in the voids, and a comprehensive compaction grouting program was completed to densify the fill. The compaction grouting layout included injection holes along four longitudinal lines, spaced at 6.5 feet across the roadway width. Within each longitudinal line, grout injection holes were spaced at 13.1 feet, and staggered 6.5 feet from the nearest point in adjacent longitudinal lines, as shown in Figure 8-11.



Haramy et al. 2012

Figure 8-11. Compaction grouting layout, Rocky Mountain, Colorado.

Injection holes were terminated at a depth in which bedrock was encountered, generally between 6.5 feet and 30 feet below the paved surface. Grout was injected from the bottom of each injection hole under consistent pressure and continued as the grout pipe was drawn from the hole in 1-foot increments.

Seismic data was collected and differences in velocity of seismic waves measured at different locations using geophones was calculated. The pre- and post-grouting seismic data was analyzed using a non-destructive volumetric evaluation method called High Definition Imaging (HDI). The study showed that the HDI method is an effective method to monitor volumetric grout propagation in near real-time (including time taken for processing of seismic data), thereby supporting its usefulness as a valuable quality assurance tool.

2.4.5.2 Case History 2: Zion – Mt. Carmel Highway, Zion National Park, Utah (Lynch et al. 2011)

Compaction grouting was used to remedy severe pavement subsidence and cracking on the Zion – Mt. Carmel Highway in Zion National Park in Utah. Damage was found to be caused by loose embankment material and voids and bottom-up compaction grouting was selected to stabilize the embankment. Subsurface drainage was improved by replacing the top 3 feet of roadway with geosynthetic-reinforced fill after completion of grouting process. Investigation of roadway structure showed an asphalt thickness greater than 3 feet resulting from repeated maintenance patching to correct subsidence and cracking problems on the pavement surface.

Grout holes for compaction grouting were drilled along the roadway in a staggered pattern at 4 feet up to a depth of 10 feet. The holes were typically about 4 feet from the pavement's outer edge. The soil was a loosely consolidated, poorly-graded fine sand with SPT values

ranging from 5 (very loose) to 14 (medium dense). Maximum slump of the grout was specified as 1.8 inches.

Two sites were selected for the project, and 79 and 107 holes required a total of 4.2 cubic yards and 15.3 cubic yards, respectively. It was observed that grouting near the surface requires additional precautions due to creation of sphere of influence around the injection point creating a “pitcher’s mound” effect, and insufficient overburden pressure to develop resistance to injection pressures. Lessons learned from this project include the need to further evaluate the use of measured slump for grouting due to the existence of a different stress state in the grout (gravitational plus pressure from pump, casing, etc.) as compared to only gravitational forces during the actual slump test.

Case History 3: Apache Trail, Tonto National Forest, Arizona (Lynch et al. 2011)

Problems faced at the Apache Trail in Tonto National Forest, Arizona were similar to those described above for the Zion – Mt. Carmel Highway. The selected grouting option was a combination of void-fill grouting and compaction grouting to stabilize the roadway fill. Grouting was performed in areas that showed evidence of void development due to piping of material within loose fill material and settlement of both the highway and adjoining retaining walls. Asphalt thickness within the pavement section varied from few inches to several feet as observed in Case History 2.

Grout pipes were installed to relatively shallow depths (2 to 17 feet). The grout mix used for Apache Trail grouting had a w/c of 0.77 by weight (1.17 by volume), and constituents as shown in Table 8-3.

Table 8-3. Grout Mix Used for Apache Trail Grouting

Constituent	Weight
Masonry sand	1,350 lb.
Top soil (silt)	1,350 lb.
Cement	432 lb.
Fly Ash	432 lb.
Retarder	24 ounces
Water	40 gallons

A thixotropic admixture and foam were added to this mix design, with amounts determined based on site conditions. The retarding admixture used was Eucon DS, manufactured by

Euclid Concrete Admixtures, and the thixotropic admixture used was Rheomac VMA 362, manufactured by BASF.

Both case histories 2 and 3 suggested the need for general guidelines for 3D seismic tomographic surveys for pre- and post-grouting highway applications. It was determined that the accuracy of seismic data increases significantly when the same locations are used for geophones and transmission points. Other factors determined to improve the accuracy of seismic velocity tomography are:

1. Use the same initial or starting velocity for model used to construct seismic tomography
2. Benchmarks for seismic velocity measurement should be at a sufficient distance from construction site.
3. Straight and parallel lines are needed for seismic line layouts
4. Maintain geophone and transmitter locations in relatively planar orientation

From the data collected during the case studies 2 and 3, performance specifications were developed to calculate pay factor as a function of the Seismic Volume Improvement Factor (SVIF), which is the ratio of average seismic velocity differences recorded from all data points to the pre-grouting mean velocity. SVIF is calculated using Equation 8-5.

$$SVIF = \frac{\sum_{j=1}^{n} \Delta v_j}{n v_{mean,pre-grouting}} \times 100\% \qquad \text{[Eq. 8-5]}$$

where,

SVIF	=	Seismic volume improvement factor
Δv_j	=	Difference in velocity at j^{th} data point between pre- and post-grouting soils
n	=	Total number of data points
$v_{mean,pre-grouting}$	=	Average of pre-grouting velocities at all data points

Improved ground conditions after grouting result in increased seismic wave velocity and hence, a higher SVIF. A possible specification pay-factor structure based on SVIF was proposed in the report, which is shown in Table 8-4.

Table 8-4. Potential Pay Factor Specification Developed from Seismic Data Analysis for Grouting

Seismic Volume Improvement Factor, SVIF (m/s)	Pay Factor
SVIF < -100	0.90
-100 < SVIF < -50	0.95
-50 < SVIF < 50	1.0
50 < SVIF < 100	1.10
SVIF > 100	1.20

2.5 Bulk Void Filling

Bulk void filling is a process where large quantities of cement-based grout is used to fill subsurface voids such as karstic cavities in soluble rock or manmade cavities such as mines. The entire cavity can be filled or low slump grout columns can be constructed to reinforce the roof of the void.

2.5.1 Applications

Bulk void filling is employed in a large array of applications, including karstic limestone cavity infill, backfilling of old mineral workings, and repair of scour problems under bridges. Many regions of the United States are underlain by limestone rock formations. Due to its solubility in water, limestone tends to erode and dissolve over time, thus forming in-situ cavities. These phenomena, also known as “sinkholes” can potentially cause the ground above to collapse or sink if they migrate to the surface. LMGs can be injected into these limestone cavities to seal the cavities and re-densify the loosened overburden soil. The rheology of the grout prevents it from flowing through the network of caverns which can exist in limestone. In this way, localized filling and stabilization of an area can be accomplished and sinkholes can be prevented. Depending on the design of the grout and the nature of the site, this approach can be adopted also in flowing water conditions (Bruce et al. 1998, Bruce et al. 2001, and Bruce 2003).

Drilling and grouting methods are commonly used to fill collapsed or abandoned coal and iron mines to prevent surface subsidence, and this has been a major application in Ohio, Pennsylvania, New Jersey, West Virginia, Wyoming, and Alabama, in particular (ASCE 2003).

2.5.2 Advantages and Potential Disadvantages

2.5.2.1 Advantages

Bulk void filling is usually an economical method of solving the problems noted above and, on many occasions, is the only viable solution when a void or cavity affects surface structures. Similar to other forms of grouting, the drilling and grouting should be considered an extension of the exploration program, while also remediating the problem. The advantages of bulk void grouting are as follows:

- Low cost per unit volume of materials when using inexpensive fillers
- Minimum disturbance of existing surface structures
- Strength of grout can be tailored to fit the in-situ condition
- Essentially yields full roof contact
- Grout can penetrate all locations without fear of the grout flowing, settling or being washed away
- Effectiveness can be verified

2.5.2.2 Potential Disadvantages

Bulk void filling commonly has two potential challenges. One is the difficulty to fill the voids completely with grout. The second is containing the grout within the zone to be stabilized, although low slump grout "barriers," accelerated grouts, and grout-filled fabric forms have been used to minimize this problem as illustrated in Figure 8-12.

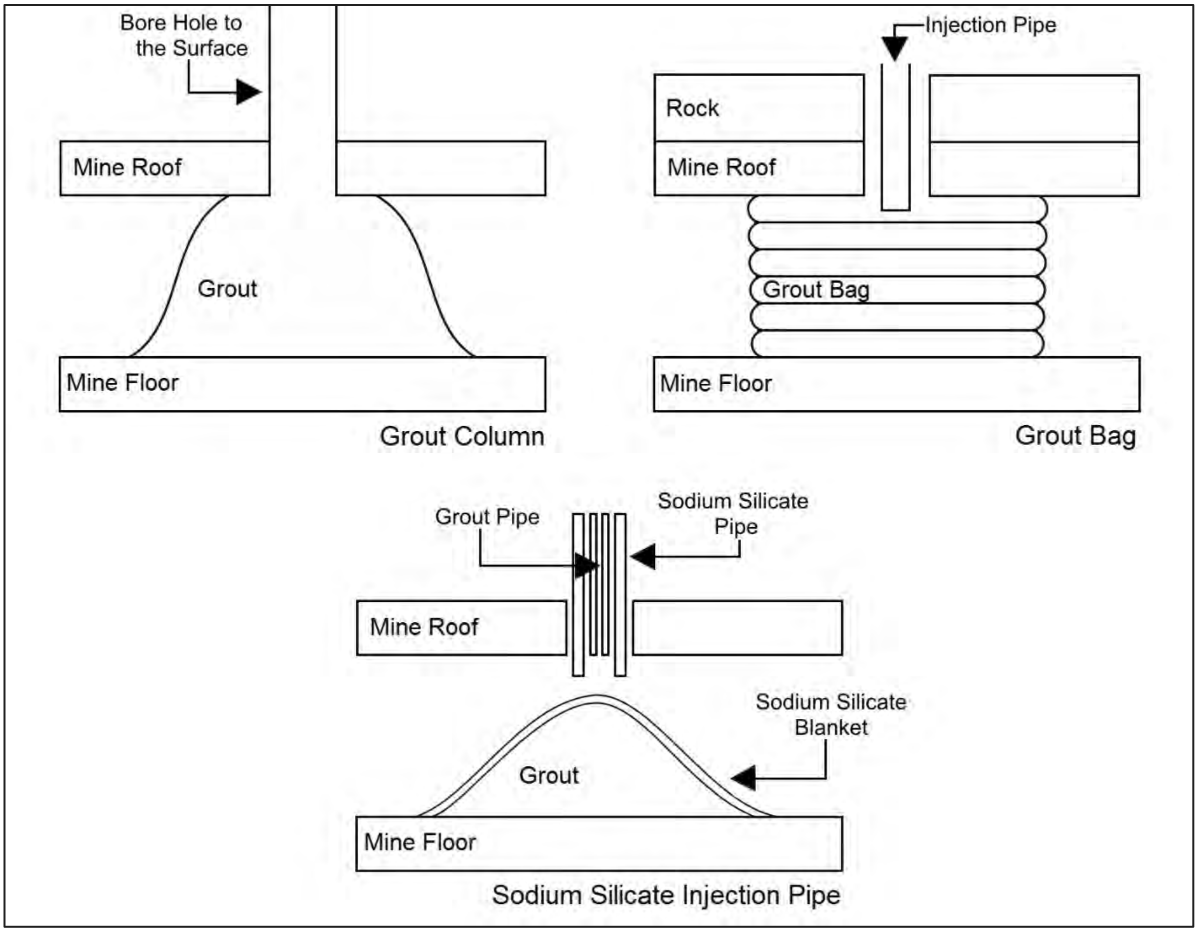


Figure 8-12. Bulk void filling methods.

2.5.3 Feasibility Evaluations

As in all grouting operations, available sources of information should be considered before the feasibility of a solution can be established. The information studied for void filling should include historical as-built data on mine and tunnel projects. Historical mine maps are an invaluable source of information, but must be related to contemporary land forms or structures. These can often be supplemented by visual surficial or underground assessments, where man-access is practical and safe. Assessment of karstic terrains is often more difficult, and may involve intensive exploration drilling, usually supplemented by a variety of geophysical (e.g., GPR, resistivity) and hydrological tests. Great variability in ground conditions may be anticipated between adjacent boreholes in karst.

As a basis for design, therefore, the lateral and vertical extent of the voids or collapsed zones must be determined together with an indication of the groundwater regime, and, in particular, if the water is flowing, where it is flowing, and at what velocity and rate. In general, it is not uncommon to identify projects involving the drilling of several hundreds of holes to depths

of greater than 300 feet and the injection of a variety of grouts formulated from hundreds of thousands of tons of materials.

2.5.4 Design Overview

This section provides an overview of design considerations for bulk void filling, grout type selection, grouting program design and performance monitoring.

2.5.4.1 Grout Types and Selection

The materials used in a void filling grouting operation can vary from non-cementitious waste materials to high-strength, low-slump concrete, depending on the purpose and intent of the project. Void filling usually encompasses one or more of the other grouting techniques and so the materials utilized in void filling vary considerably. The sections corresponding to these different grouting techniques should be reviewed when considering a void-filling project (i.e., LMG, fissure grouting, and HMG).

When filling scoured zones with concrete filled tubes or bags, a fine aggregate concrete (structural grout) is recommended. The typical range of mix proportions is shown in Table 8-5.

Table 8-5. Typical Range of Material Mix Proportions for Void Filling Applications

Material	Mix Proportions (lb./cubic feet)	Mix Proportions (kg/m^3)
Cement	37 – 47	600 – 750
Fly ash	11 – 14	180 – 220
Sand	134 – 144	2150 – 2300
Water	33 – 37	525 – 600

2.5.4.2 Design Procedure/Program

General good grouting practices can be used to completely fill voids in the ground. However, if clay or other erodible material is present as infill, then it is best to remove as much of this material as possible prior to grouting. Removal can be achieved by flushing with air and/or water and/or dispersant. Clay trapped in grouted karstic cavities can be removed if subjected to prolonged differential head.

It is important to realize that the extent of a cavity is unknown after penetration by just one grout hole and even the thoughtful implementation of appropriate geophysical techniques

may not yield conducive information. Sometimes, it may be necessary to use intermittent grouting, which is the process of injecting grout in the hole and then waiting several hours before injecting additional grout. In practice, the maximum quantity of grout to be injected varies from about 30 to 1,000 cubic feet or more per injection period. A limit may also be placed on the maximum amount of grout to be injected into a single hole. This practice differs from that recommended for fissure grouting practice.

When grout injection refusal is reached, it is assumed that grout has filled at least the portion of the cavity penetrated by the grout hole. Additional grout holes are then drilled and grouted until the desired results are achieved in a split spacing sequence. If pressures fail to build up or the cavity is too large to grout in this manner, grouting should continue with a grout curtain placed to control the flow of grout from the cavity, or radically different materials and methods should be considered. Additional exploration, consultation, evaluation, and design of treatment can then take place without delaying the project. These measures may call for specialized grouting procedures or materials, such as foaming agents or accelerators, positive cutoff diaphragms or formed concrete wall, hot bitumen, additional excavation, grout filled bags, or some other solution. Grout hole spacings and locations will be dictated by the site conditions, but holes on a final grid spacing of 10 feet or less are not unusual for “tightening” purposes.

2.5.4.3 Performance Monitoring

Bulk void grouting involves drilling several holes adjacent to each other prior to grouting. Borehole cameras are available that can be placed in adjacent drill holes to observe and verify that the injection of grout is satisfying project requirements. Instrumentation can also be specified to monitor heave, settlement, etc. during the grouting program while close analysis of grout volumes and pressures attained during each phase of grouting remains the classic performance monitoring technique. Appropriate geophysical methods may also be of value.

2.5.5 Cost Data

In most grouting projects to fill voids, overburden materials and rock must be penetrated to reach void elevations. Normally, a primary, secondary, and sometimes tertiary hole spacing is utilized. The primary grout hole grid pattern may range from 10 to 100 feet on center. The diameter of the drill hole normally ranges from 3.0 to 8.0 inches and cost starts at $7.00 per linear foot. The cost for supplying, mixing, and injecting the grout normally ranges between 57 to $153/cubic yard. A review of costs for bridge scour repair using concrete fabric forms from 1968 to 1976 in Pennsylvania indicates a range of $230 to $765/cubic yard (Okonkwo et al. 1998).

The cost of void filling projects comprises:

- Mobilization and demobilization
- Drilling (production and exploratory)
- Flushing and water testing
- Mixing and injecting grouts
- Materials
- Verification drilling and testing

The mobilization/demobilization cost will vary, based on the complexity and number of drill rigs and grout plants required. The mobilization of a single drill and grout plant should be under $15,000.

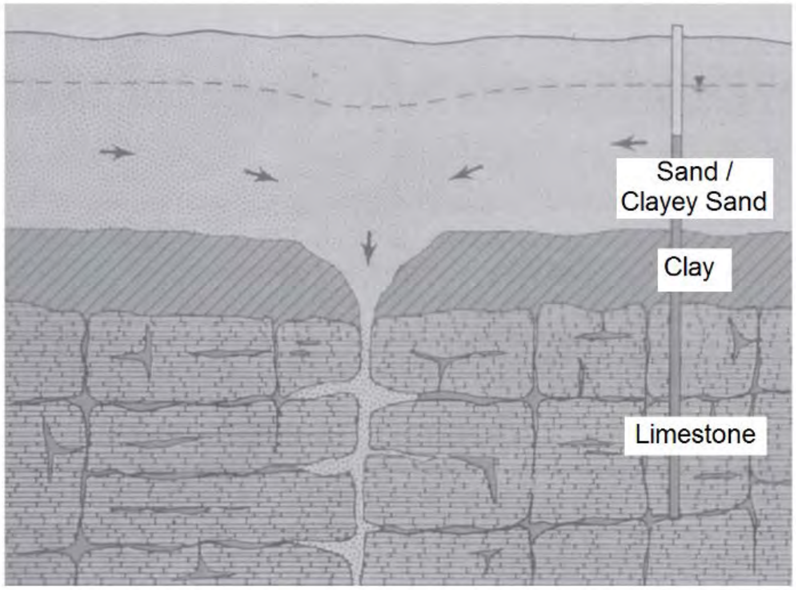
2.5.6 Case Histories

Two case histories are presented where bulk void filling was used to fill subsurface voids. The third and fourth case histories presented in this section describes the use of bituminous grout and cemented paste for mine backfill.

2.5.6.1 Case History 1: Sinkhole Remediation in Hillsborough County, Florida (McGillivray et al. 2012)

A grouting program was developed by Hillsborough County, Florida, to remediate road failures caused by sinkholes. Karstic limestone is the major terrain type in the county, which undergoes formation of several sinkholes due to erosion of the sandy overburden into voids in limestone. The County, in collaboration with an Engineer-Contractor team developed a rapid response system to investigate, remediate, and manage sinkhole grouting projects. Costs for investigation, pipe installation, and grouting were established by the County for selecting the team to eliminate delays.

Typical layer profile in this area of Florida consists of 30 to 60 feet of sand or clayey sand, followed by a 5 to 20 feet stratum of highly plastic clay and a very porous weathered limestone layer underneath as shown in Figure 8-13.



McGillivray et al. 2012

Figure 8-13. Typical soil layer profile in Hillsborough County, Florida.

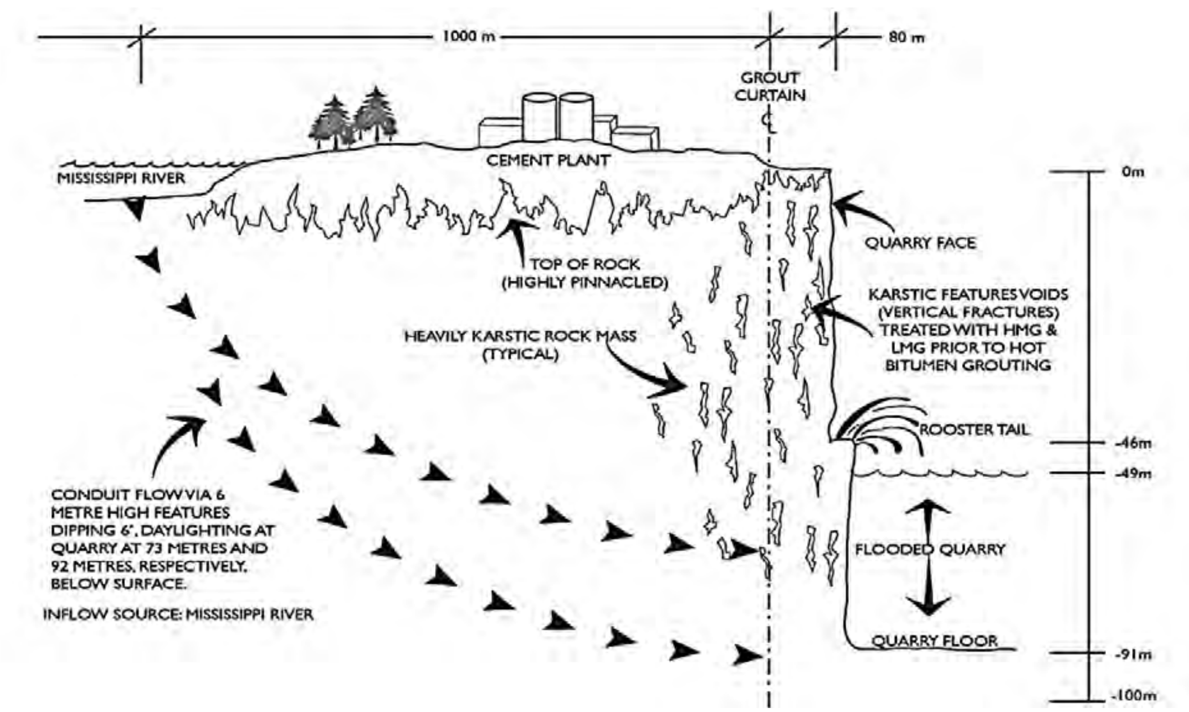
Remediation of sinkholes in Florida was typically accomplished by installing grout pipes within and around the depression/drop-out, and pump a sand/cement/fly ash grout in the soil above the sinkhole. The purpose of grout is to prohibit vertical seepage and compact soils disturbed by the sinkhole formation and fill any open voids left in the soil. Sinkholes were identified from loss of drilling fluid, which is indicative of vertical seepage that causes soil erosion into limestone.

Grout holes for the sinkhole remediation project were drilled into the limestone layer at 5 feet from each other in three rows extending beyond the sinkhole limits. The treatment required 13 pipes for the sinkhole at depths ranging from 46 to 77 feet. Cost of grout for a highway sinkhole remediation project in 2003 that required 408 cubic yards of grout was $45,000, which was about two-thirds of the total project cost. Later, synthetic foam characterized as Controlled Low Strength Material (CLSM) was added to bulk fill grouts with typical cement grout-foam ratios of 60 to 40. Mixing foam with grout resulted in cost savings of about 20 to 25%. The foamed grout strength was measured in the laboratory as 290 psi as compared to ~4000 psi of a typical grout, but pumped relatively easily at low pressures.

2.5.6.2 Case History 2: Hot Bitumen Grouting, Lonestar Quarry, Missouri (Bruce and Chuaqui 2012)

Hot bitumen in conjunction with LMG and HMG was used to stop high magnitude inflow of water into Lonestar Quarry in Cape Girardeau, Missouri. The quarry geology is heavily karstic with several large cavities, and the hydraulic gradient increased with depth of

excavation. The source of inflow was determined to be the Mississippi river located about 3280 feet from the quarry from which two conduits measuring almost 20 × 30 feet, which were centered at 250 feet and 305 feet below grade, carried a 35,000 gallons/minute inflow to the quarry floor, which was more than 330 feet below grade as shown in Figure 8-14.



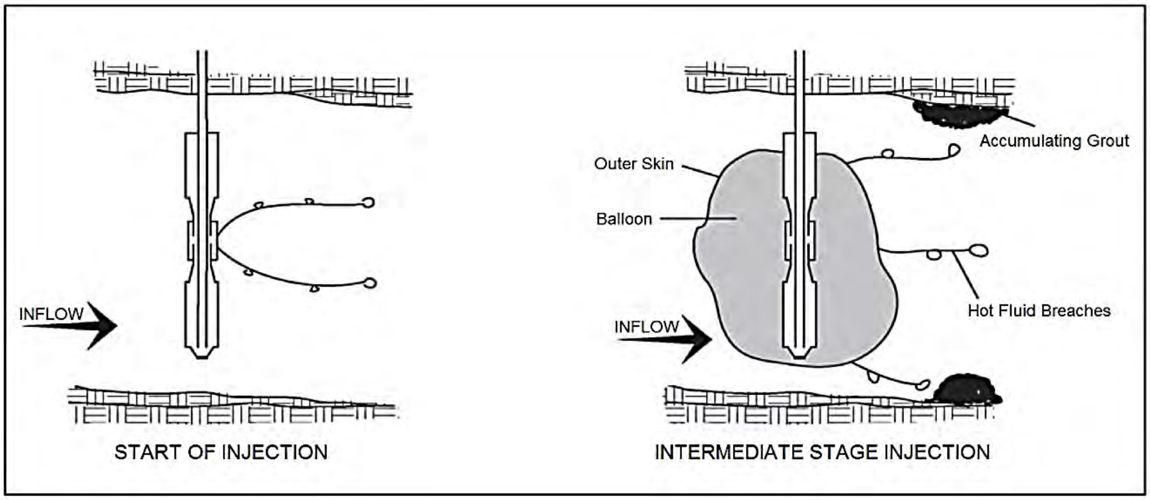
Units: 1 m = 3.28 feet or 1.09 yards

Bruce and Chauqui 2012

Figure 8-14. Illustration showing conduit inflow into Lonestar Quarry.

A small leak led to massive flooding within two weeks, when the inflow was eliminated by injecting hot bitumen along with low- and high-mobility cement grouts. This case history is not related to highway problem mitigation, but is nevertheless interesting due to the use of hot bitumen for waterproofing under high flow conditions. Hot bitumen in grouting was also used to seal a massive water flow in karstic lime in Florida (Bruce et al. 2001).

The advantage of using hot bitumen over cement based grouts is that it is easily pumpable in its liquid state at temperatures above 400°F, and the curing mechanism is driven not by a time-dependent chemical reaction but temperature rate of cooling. The hot melt therefore cooled down instantaneously before getting diluted or washed away. Initial injection of bitumen was done upstream of location where grouting was desired, and the hardened bitumen gushing out of the injection point cooled down and adhered to the walls of the pathway as shown in Figure 8-15.



Bruce and Chauqui 2012

Figure 8-15. Bulk void filling using bitumen, Lonestar Quarry, Missouri.

Vertical holes were drilled into the rock within a length of 20 feet to directly intersect the conduit flow in order to locate the most effective bitumen, LMG and HMG injection points. The arrangement of drilled wells was not pre-determined but evolved as information regarding the conduit pathways was confirmed or new information discovered during each phase of drilling. HMG wells were drilled 10 feet upstream of the bitumen injection wells.

This project demonstrated the use of highly specialized grouting applications to overcome problems caused by high groundwater flow rates. The principle of bitumen solidifying upon reduction in temperature was used for this project, which is also the greatest disadvantage against its use. Care must be taken to prevent freezing of bitumen within the delivery tubing by maintaining a high exit temperature prior to injection by using down-hole pre-heating.

2.5.6.3 Case History 3: Ground Seepage Cutoff in Karstic Limestone – West Virginia Limestone Quarry (Walz et al. 2003)

A large operational dolomitic limestone quarry is situated in West Virginia less than 1,500 feet from the Shenandoah River. In April 1997, a major sudden inflow developed into the southwest corner of the quarry pit following production blasting activities and several abnormally severe precipitation events that caused flooding of the river and nearby sinkhole formation. An observed vortex in the river appeared to be the point source of the flow. The initial magnitude of the flow, estimated at over 35,000 gallons/minute was far greater than the capacity of the existing pit pumping facilities.

The new inflow posed a severe threat to both the current and future viability of the quarry. Several unsuccessful attempts were made to construct a cofferdam with sandbags on and

around the location of the vortex. In May 1997, pumping operations were discontinued and the quarry water level was allowed to rise. Extensive investigations were conducted to determine the source and extent of the inflow. Prior to the design and construction of the remediation, it was agreed to "baseline" the hydrogeologic situation as closely as possible. Wells with deep piezometers were located between the river and the quarry to evaluate water level, pH, conductivity and temperature. Monitoring continued during and after remediation.

The owner's goal of the remediation program was to reduce the total inflow into the quarry to a flow of 8,000 gallons/minute with the quarry completely de-watered. Later data would indicate that this would require reducing the flow from the river to below 3,000 gallons/minute. Three specific options were considered:

1. Identify the specific solution cavities in the river and seal them
2. Construct an intercepting cut off at some appropriate location between the river and quarry
3. Treat the problem close to the quarry

Option 2 was clearly favored on logistical, technical, and environmental grounds, and it was decided to locate the cutoff on a convenient roadside location about 50 feet from the riverbank.

The main challenges faced during the design of grouting program were:

- Very high velocity and rate of flow through potentially multiple conduits
- Mud filled karstic features, creating the possibility for erosion
- Piping and "blow out" after curtain placement when the hydraulic gradient increased
- Possibility of grout migration "upstream" into the river

Several grouting technologies were studied to provide the curtain such as, in part or in whole, jet grouting, polyurethane injection, LMG, hot bitumen injection, accelerated cement-based slurries (HMG), use of the multiple packer sleeve pipe (MPSP) system, and geotextile grout-filled bags. For the very severe geological and hydrogeological regimes to be accommodated, each technique was assessed based on technical feasibility, likelihood of successful treatment of the inflow in both short and long terms, and cost. Grouting was accomplished in nine phases. Throughout the grouting operation, several modifications were made to enhance control and responsiveness and allow simultaneous injection of both bitumen and slurry into the same hole. For example, stringers were used to allow the simultaneous injection both slurry and bitumen into the hole. It was decided to first treat the "Cold Karst" zones (open voids without flowing water) with LMG and slurry grout via the MPSP system and then treat

the “Hot Karst” (zones with flowing water) with hot bitumen from the downstream row of holes, backed up by slurry grouts simultaneously injected from the upstream row via further MPSP locations.

Monitoring of groundwater wells, water levels in the quarry, flow, and visual observations of the river eddy indicated that the program was successful. By the end of the grouting, the flow from the river into the quarry had essentially stopped. The success of this case history illustrates many important features of which three are particularly noteworthy. First, the study is an illustration of how contemporary grouting technology, when correctly designed, implemented, analyzed, and closely monitored can be successfully used even in the most adverse conditions. Secondly, all sources of information must be studied before and during the operation in order to gain the best possible picture of changes in the geological regime brought about by the grouting. Thirdly, the project illustrated the need for all stakeholders (owners, designers, consultants, and contractors) to partner fully and openly, and provide mutual support at all times and in all aspects to ensure successful completion of work in arduous and stressful conditions.

2.5.6.4 Case History 4: Coal Mine Remediation in South Central Illinois (Greenwood and Hill 2012)

Subsidence is a severe problem in underground mines due to collapsing of old mines. Low and high mobility cementitious grouts were injected into an abandoned coal mine in South Central Illinois to completely fill the mine. Down-hole cameras and lighting were discussed to increase the accuracy of estimating grout quantities and scheduling void fills in unmapped, inaccessible mines. Typical subsidence in the area results in surface settlements of 12 inches to 39 inches over depressions as large as 300 feet in diameter, which is also reflected in surrounding structures such as pavements and floor slabs.

Filling the mine void with cementitious grout has been successfully used in South Central Illinois to reduce subsidence. The approach consists of creating a grout barrier using a stiff, low mobility cement-based grout (LMG) with a slump of 2 to 4 inches around the perimeter up to a pre-determined distance from the affected structures. The barrier consists of primary and secondary holes at 10 to 20 foot spacing. A high mobility, low viscosity grout (HMG) is used to infill the space within the barrier, which is intended to spread through the mine provided the mine voids are continuous and unobstructed by debris.

Grout mixes are prepared from cement, fly ash, bentonite, and water, with different rheology for barrier and infill grouts. Compressive strength is usually designed to exceed the typical overburden pressure of 500 psi, while injection pressures are delivered mostly from gravity head. The cutoff pressure used in the project was 21 psi measured at the pressure gage head,

or grout is rejected during the injection process. Supplemental grout columns using LMG were also constructed to reduce the pressure on existing coal pillars as shown in Figure 8-16.

Greenwood and Hill 2012

Figure 8-16. Supplemental column grouting in an abandoned mine, south central Illinois.

Quality control was achieved by sampling and testing grout materials, test drilling, and downhole video camera work. Camera exploration provides information about area width and is used to verify accuracy of the mine map. Real-time data pertaining to flow rate, specific gravity, and pumping pressures was also obtained for monitoring purposes. The use of computerized data collection systems greatly improved the efficiency and quality of mine grouting.

2.6 Slabjacking

Slabjacking or slab stabilization is a grouting technique used to restore base/subgrade support to concrete pavement slabs by filling the underlying voids with foam or grout. Voids develop underneath the concrete slab at joints, cracks, or the pavement edge due to pumping or consolidation of the subgrade caused by repetitive heavy traffic loads (Fung and Smith 2010). Slabjacking is sensitive to construction practices and care should be taken when injecting foam or grout, so as not to cause other problems such as accidentally sealing the transverse joints.

2.6.1 Applications

Slabjacking is used to correct settlement of concrete slabs formed over soils, such as organic soils, compressive clays, and silts that have consolidated, or materials that have been washed or eroded away. This application is especially appropriate for highway maintenance and preservation activities (Welsh 1997). Slabjacking procedures include raising or leveling, under-slab void filling (no raising), grouting slab joints, and asphalt subsealing. Most slabjacking uses a suite of cementitious grouts, incorporating bentonite, sand, ash, and/or other fillers as dictated by local preference and the project conditions and goals. Certain proprietary methods use expanding chemical foams to create uplift pressures. Best results (when no cracking is caused to the slabs) are obtained when the slabjacking is uniformly and gradually conducted. Slabjacking can also be used to “pump” at expansion joints that have sunk below the adjoining section.

A 1977 study, Slabjacking-State-of-the-Art summarized the various slabjacking practices then employed by State Transportation agencies as follows (Committee on Grouting of the Geotechnical Engineering Division 1977):

1. Slabjacking (raising or leveling) - 25 states.
2. Under-slab void filling - 17 states.
3. Grouting slab joints - 6 states.
4. Subsealing (hot asphalt) - 3 states.
5. Filling voids prior to overlay - 6 states.

2.6.2 Advantages and Potential Disadvantages

2.6.2.1 Advantages

The advantages of slabjacking include the following:

- It is frequently the most economical repair method.
- It is usually faster than other solutions, especially compared to removal and replacement.
- It can be planned so that there is little disruption to the existing facility, and can be performed at times of light or no traffic.
- The equipment needed to perform the slabjacking operation can be remote from the repair location, providing for maximum accessibility.
- Increased load capacity of the slab is provided.

- The service life of the concrete pavement is extended by reducing deflections.
- A smoother riding surface is established.
- It is most effective when pavement has undergone minimal structural damage.

2.6.2.2 Potential Disadvantages

Slabjacking has the following disadvantages:

- Cracks already present may tend to open up when the slab is treated, unless care is taken with the process.
- Slabjacking may not be cost-effective on small projects.
- Slabjacking may not address the original cause of the settlement.
- Slabjacking is not very effective on pavements that have already undergone significant structural damage (distresses).

2.6.3 Feasibility Evaluations

When a slab or structure has settled differentially, a cost analysis is key in determining whether to replace the slab and correct the cause of the problem or to jack the slab back to its original elevation and repeat this process periodically. Slabjacking is typically not appropriate where the cracking is severe. Local contractors can be contacted to provide budget estimates and feasibility studies.

2.6.4 Design Overview

This section provides an overview of design considerations for bulk void filling and slabjacking, grout type selection, grouting program design, and performance monitoring.

2.6.4.1 Grout Types and Selection

Most slabjacking can be successfully completed using a grout composed of Portland cement, fine sand, and water, although bentonite and chemical admixtures may be used to provide appropriate rheological properties. Cement content varying from 5% to 10%, depending upon the sand gradation and admixtures, will be sufficient to provide a grout strength in excess of 480 Pa. Where higher strengths are needed, higher proportions of cement can be used. Water content should be adjusted to provide the necessary consistency. Ideally, sand material should be well graded, with 100% passing a #8 (2.36-mm) sieve, with not more than 20% finer than 0.002 inches. Calcium chloride or high early strength cement can be used to accelerate the set, and admixtures that can control the shrinkage or expansion can also be added. Where exceptionally high strengths are needed or excessively coarse sands must be

used, admixtures are generally not used. In these cases, pozzolan (15 to 50% of the weight of the cement) will improve the pumpability of the grout.

2.6.4.2 Design Procedure/Program

Prior to undertaking any slabjacking program, the underlying cause of the problem and the desired end results must be determined. If slabjacking is used for settlement re-leveling, future leveling may be required. If the roadway pavement that is to be stabilized is to receive an overlay, virtually no lifting may be required. Regardless of the cause of the problem, the engineer should accurately specify the necessary performance requirements and tolerances for the project.

Another consideration is the appearance of the finished surface. Most slabs that have settled contain at least some cracks. Although slabjacking can be performed without creating new cracks, those cracks already existing will be visible.

Slabs restored by slabjacking will contain patched injection holes usually on a grid of 5 to 6 feet. Therefore, the surface finish conditions should be considered in advance of the work. These factors will vary depending on the affected facility. While minor defects may be tolerable on a highway, they will not be acceptable on a tennis court (although such applications are remarkably few in number).

2.6.4.3 Performance Monitoring

The objectives of slabjacking are to fill voids and raise the slab to its approximate original elevation, without causing additional damage to the slab. Instrumentation as simple as a string line can ascertain this objective, although the use of lasers results in a more accurate monitoring of the grouting process.

2.6.5 Cost Data

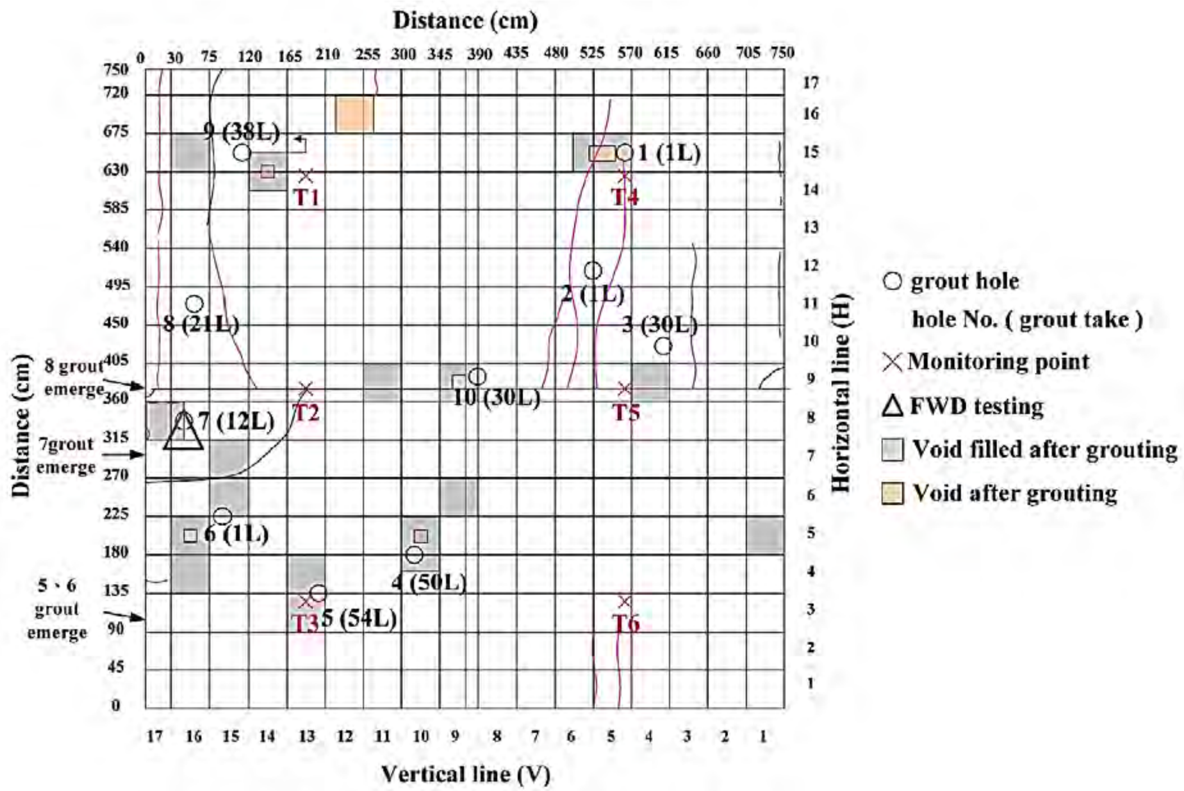
Due to the extra effort involved in delicately raising a slab, the unit cost is normally higher than for only filling voids beneath slabs. For estimating purposes, a cost of $300/cubic yard of grout injected is a good starting point. Slabjacking using polyurethane may be estimated at $70 to $100 per square yard of slab raised up to 2 inches.

2.6.6 Case Histories

2.6.6.1 Case History 1: Trial Grouting Under Rigid Pavement in Magong Airport, Penghu, Taiwan (Ni and Cheng 2012)

A trial grouting program using cementitious grout was undertaken to remediate settlement under Portland cement concrete (PCC) slabs in Magong Airport. Laboratory testing was conducted to determine a suitable cement-based grout mix, and the mix having a w/c ratio of 0.8 and an additive of 7% by weight of cement was selected as having the desired stability, flow, initial setting time, shrinkage, and strength. A circulating grout injection system (shown in Figure 8-36) was used to control the injection pressure at the grout header, and the pressure increment over initial contact pressure was kept at lower than 7.1 psi to prevent uncontrolled slab heave.

Slab elevations were monitored using displacement gages (LVDT) mounted on two reference beams during the grouting process. Grouting trials were conducted on four different slabs. Voids under the slab were identified using Ground Penetrating Radar (GPR). Falling weight deflectometer (FWD) testing was conducted to evaluate the effective of grouting program to restore the load carrying capacity of the slabs. Figure 8-17 shows the grouting and monitoring holes, location of filled and unfilled voids after grouting, and FWD test locations. FWD test results showed lower deflections for all four slabs, indicating an increase in the load carrying capacity of the slab.



Units: 1 cm = 0.4 inches

Ni and Cheng 2012

Figure 8-17. Results from concrete slab 3 monitoring, Magong airport.

The total volume of grout required to fill the voids was greater than the volume calculated as the product of slab area and maximum allowed heave of 0.04 inches. This difference was attributed either to large void thickness or permeation of grout into the subbase. Drilled holes were cleaned of debris and filled with quick setting patch material after completion of the grouting program.

2.6.6.2 Case History 2: Injected Polyurethane Slabjacking for Concrete Slab and Bridge End Panel Stabilization, Oregon (Soltesz 2002)

Slabjacking was used to stabilize a bridge end panel and an adjacent cement concrete slab in Oregon using the URETEK^{TM} method. This method uses a high density polyurethane created using a two-compound system. The first compound consists of a mixture of polyhydroxy compound, catalysts, and water, the second being an isocyanate compound. Initial profiling of the bridge and roadway was conducted and 0.63-inch diameter holes were drilled through the pavement into the soil below. Formation of the final polyurethane foam from constituent compounds leads to a large volume increase that lifts the slab upwards.

Advantages of using polyurethane foam for slabjacking are as follows:

- Faster setting time – foam achieves up to 90% of full compressive strength within 15 minutes of injection
- High compressive and tensile strengths
- Expansive material requires fewer and smaller injection holes
- Lightweight foam does not contribute to subgrade settlement
- Low water infiltration due to closed cellular structure of foam

The slabjacking was designed to lift the concrete slab by 3.5 to 4 inches. Six holes of depth 20 inches were drilled in various locations and URETEK™ 486 polyurethane form was injected to raise the slab to the desired profile. After foam injection, additional holes were drilled at 4 feet spacing all over the slab to fill pre-existing voids or voids formed during the initial injection process. All holes were finally sealed with a cementitious grout. The entire operation took 10.5 hours to complete at $9.10 per lb. and a total cost of $42,260 for 4649 lbs. of injected material.

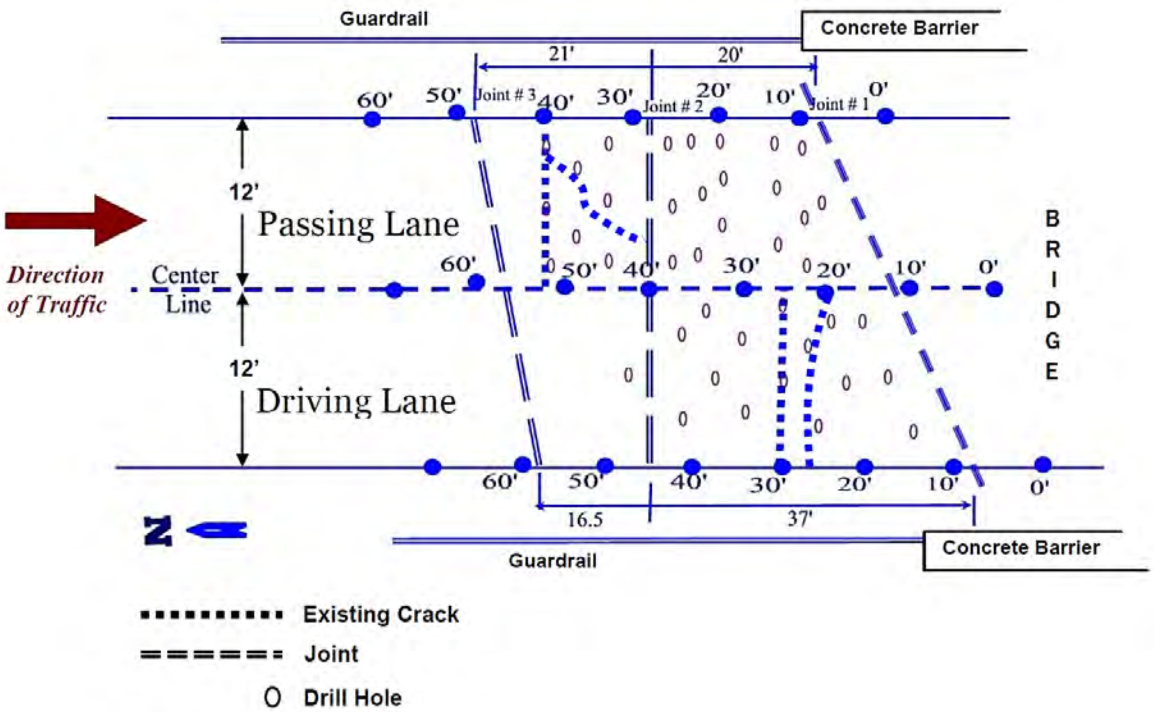
Stability of the Glenn Jackson Bridge site and water permeability of foam was monitored for two years after the grouting activity. Monitoring was performed by drilling surveying holes into the slab at both approach and leave ends, and data collected after 3, 6, 12, 18, and 24 months was compared to baseline measurements taken 4 days after the slabjacking. Settlement measured after 24 months varied from 0.08 inches to 0.4 inches, with most of the settling occurring within the first three months. Two sets of grout samples were obtained which showed different densities (low vs. high), with the low density field sample having a much lower average compressive strength (47.5 psi) than the high density sample (97.3 psi) after 23 months. It was also observed from laboratory testing that the compressive strength of polyurethane foam did not decrease after exposure to field conditions.

2.6.6.3 Case History 3: Wisconsin Department of Transportation Report – Slabjacking Study on Interstate Highways (Abu and Labarca 2007)

The Wisconsin Department of Transportation (WisDOT) investigated the URETEK method of slab jacking to correct differential settlement of concrete pavement bridge approach slabs. Two test sites were selected and the functionality of the slab jacking method and stability of the slabs after pavement lifting were monitored over a five-year period. Literature reviewed in the study identified several successful applications of the technology in Louisiana (Gaspard and Morvant 2004), Michigan (Opland and Barnhart 1995) and Oregon (Soltesz 2002).

The procedure involved identifying several station locations and injecting high-density polyurethane form under the slab through 5/8-inch holes drilled in the concrete. The first test

site was located on I-39 in the Southwest Region at the interchange of I-39 and US 78. Maximum elevation of slabs required was 1.5 inches, with additional holes drilled for grout injection at stations where no slab rise was observed. The total quantity of grout used in the two driving lanes was 3,240 lbs. which was five times higher than the contractor's estimate of 600 lbs. The plan of pavement section and location of grout holes on I-39 is shown in Figure 8-18.



Abu and Labarca 2007, Wisconsin Department of Transportation

Figure 8-18. WisDOT slab jacking, I-39 grout holes layout.

The second test site was located in the Southwest Region on US 12, where grouting work was performed in the left and center lanes of on approach slabs of the three-lane highway. Drilling was assisted by the County forces to speed up the drilling and injection process, and no loss of materials through the joints. The total grout quantity used at the site was 1,043 lbs., which was twice the estimated quantity of 550 lbs. The use of significantly higher grout quantities indicated the need for Ground Penetration Radar (GPR) analysis prior to pavement lifting in order to better estimate the nature and size of voids underneath the slab.

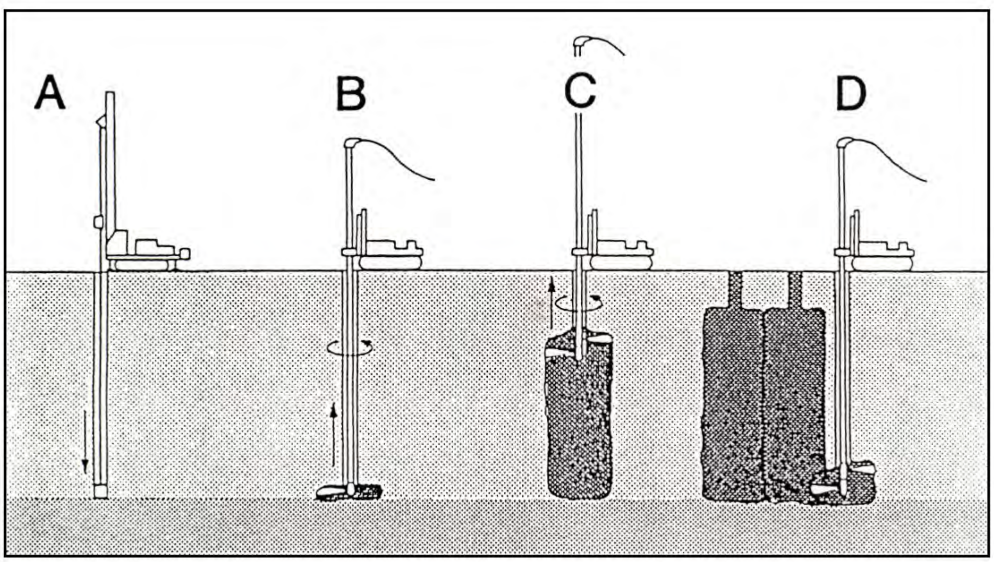
Cost analysis for this study showed that the use of URETEK™ ($243/sq. yard for I-39 and $117/sq. yard for US 12) was less expensive than total slab replacement($425/sq. yard), but more than HMA overlay ($45/sq. yard for I-39 and $63/sq. yard for US 12) and mud-jacking. Inspection of the slabs on I-39 after five years showed slight re-settlement of the test slab, but better ride quality than prior to slab lifting. Ride quality on US 12 was found to be adequate

with no additional cracks in the approach slabs. It was concluded from the study that foam injection is a practical solution to correct differential slab settlements, and is economically not viable for filling voids below the pavement.

2.7 Jet Grouting

The jet grouting technique employs high pressure, high velocity erosive jets of water and/or air to break down the soil structure, removing varying proportions of soil and mixing and replacing them with a cement-based grout. Soil particles not removed become mixed with the grout in-situ to form a treated mass. The combination of sophisticated equipment, more extensive technical knowledge, and successful applications has made jet grouting a successful ground treatment technique, compatible for use with almost any soil type from sands and gravels to highly sensitive clays.

The different types of jet grouting are intended to transform soils into a mixture of soil and cement, typically referred to as "soilcrete." Jet grouting permits the shape, size, and properties of these treated masses, usually circular columns, to be engineered in advance with an increasingly high degree of precision as illustrated in Figure 8-19.



A Drilling
B Erosion and mix-in-place operation
C Development of column-like element
D Completed elements forming a wall-like structure of interlocking elements

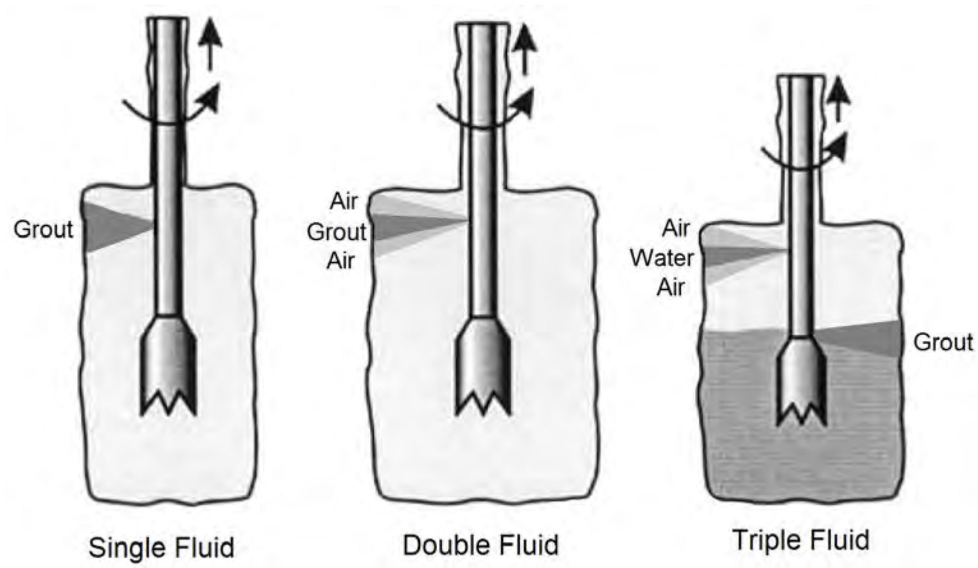
Kutzner 1996

Figure 8-19. Jet grouting process.

However, design of columns is still dependent on the soil properties. Columns are created by injecting cement-based grout at high velocities at the base of the drill string to create an in

situ soilcrete. As the drill string is withdrawn from the soil and rotated slowly, the grout jet cuts and mixes the soil for a finite distance around the drill string forming column elements. The erosive/mixing action of the jet of grout can be enhanced by various systems, which use supplemental simultaneous application of high pressure air and/or water in addition to the grout jet.

There are basically three distinct types of jet grouting, as shown schematically in Figure 8-20 (Bruce 1994, Wheeler and Burke 2000).



Wheeler and Burke 2000

Figure 8-20. Jet grouting: single (left), double (middle), and triple (right) fluid systems.

The three types of jet grouting are as follows:

- One-Fluid System: The fluid is grout, and in this system, the high-pressure jet simultaneously erodes and injects. It involves only partial replacement of the soil.
- Two-Fluid System: This method uses a high-pressure cement jet inside a compressed air cone. This system results in a larger column diameter than the one-fluid system, and provides a higher degree of soil replacement, although often lower strength than the three-fluid system.
- Three-Fluid System: An upper injection of high-pressure water inside a compressed air envelope is used for excavation and a lower jet (usually at lower pressure) emitting grout to replace the slurried soil. This system typically has a higher degree of soil replacement than the one- or two-fluid systems.

2.7.1 Applications

2.7.1.1 Water Control

Jet grouting has demonstrated its effectiveness in both horizontal and vertical water control under static water conditions, as the grouted mass is generally less permeable and stronger than the in-situ soil. It can also be used in contaminated soils provided appropriate precautions are taken with cleaning equipment, protecting personnel, and disposing of spoils.

2.7.1.2 Settlement Control

Jet grouting has been used to provide foundation support through weaker, soft soils to more competent bearing strata, by increasing the strength of the weaker soils.

2.7.1.3 Underpinning

Jet grouting has become a viable alternative to conventional underpinning since its introduction into the United States in 1986. Since jet grouting can serve two purposes, as both an underpinning element and as an excavation support, it can have a considerable economic advantage. Jet grouting is a comparatively safe operation; construction personnel are never required to work beneath the structure being underpinned, and there is no need to make load transfer connection between the existing foundation and the underpinning units.

2.7.1.4 Scour Protection

Jet grouting has proven to be an effective means of providing scour protection around bridge piers and marine works.

2.7.1.5 Excavation Support

Jet grouting can be conducted immediately next to and through the footings adjacent to the excavation, allowing for a vibration-less, safe, and designable method of excavation support. Jet grouting can also be used to place excavation cross bracing prior to excavation, so that inward deflection of the excavation support is prevented. Steel reinforcement can be placed in the soilcrete to enhance axial and lateral capacity.

2.7.1.6 Liquefaction Mitigation

Jet grouting transforms potentially liquefiable soils into a cemented mass or can create a “cellular structure or honeycombs” to stiffen the soil mass.

2.7.1.7 Treatment of Karst

Jet grouting has been used to remove clay from karstic features and replace it with engineered grout (Bruce 2003).

2.7.1.8 Remedial Treatments – Gaps in Retaining Walls

Grout columns produced by jet grouting have been used to improve the strength or reduce permeability of soils at retaining walls.

2.7.2 Feasibility Evaluations

2.7.2.1 Geotechnical

Jet grouting technology can treat soils ranging from clays to gravel. Best results with jet grouting occur in cohesion-less or soft cohesive soils. The grouting process can be performed above or below the groundwater table, and has been installed to depths greater than150 feet, although common applications are less than 100 feet deep. Use of jet grouting in highly plastic soils and fibrous peat that are less erodible is not recommended unless particular action is taken such as pre-cutting the materials with only water. Also, very coarse or open graded soils will permit the grout to travel considerable distances and hence, leave the zone intended to be treated. The strength of soilcrete is reflective of the amount of cement added and of the initial soil type and consistency – sands and gravels give a higher strength and result in a more homogeneous soilcrete than silts and clays.

2.7.2.2 Environmental

The quality of soilcrete produced by jet grouting is affected by presence of organics or very low pH in the groundwater, or flowing groundwater (Burke 2012). Improvement in efficiency of jet grouting technologies has led to reduction in carbon dioxide emissions from about 3.75 lb/cubic feet (60 kg/m3) for triple-fluid jet grouting to less than 0.63 lb/cubic feet (10 kg/m3) as reported by Yoshida (2010). Jet grouting of soil produces spoil due to erosion, which can be re-used or integrated on earth works (Pinto et al. 2012).

2.7.2.3 Project Conditions

Jet grouting is particularly well suited to any area that has a high density of structures or utilities, where the ground is very variable, or otherwise not amenable to other grouting techniques, and where significant strength (say over 435 psi) is required from the treated soil mass.

2.7.3 Design Considerations

When jet grouting is used for underpinning and excavation support, three- to five-foot-diameter columns are typically designed. Construction of the columns is sequenced such that no more than three feet of temporary bridging is required from the existing foundation. The treated soilcrete strengths for the single-fluid system in Table 8-6 can be used as a guide to evaluate the design feasibility of the underpinning and/or excavation support operation.

Table 8-6. Range of Typical Soilcrete Strengths – Single-Fluid System

Soil Type	Soilcrete Unconfined Compressive Strength, psi
Clean sands and gravel	750 to 1,250
Silts and Silty sands	500 to 750
Clays	250 to 500
Organic silts and peats	less than 250

Source: Burke 2004

Regardless of system (single, double or triple fluid) the strength is a function of the cement content in the final product. The triple fluid system typically has higher replacement, so there is more cement and hence higher strengths. Double-fluid strengths may occasionally be lower due to the air entrainment, although the details of the individual site and the contractor's means and methods will govern. Acceptance of the grout is subject to at least a minimum number of jet grout samples exceeding a minimum 28-day unconfined compressive strength, both of which are determined by the engineer (ASCE 2009).

Excavation support and underpinning applications should be designed using standard design procedures, taking into account the loads that will be transferred through the foundation being underpinned by the jet grouted It should be emphasized that the final strength of the soilcrete will depend on the nature of the in-situ soil, and the contractor equipment, means and methods. Therefore, it is essential to conduct a production field test to confirm the actual column size, shape, verticality, homogeneity, and strength can be achieved. Design values should be restricted to ≤ 50% of the ultimate strength values (f_c) to accommodate inherent soilcrete variability.

2.7.3.1 Spacing

Jet grouted columns can be in the range of 2.5 to 15 feet in diameter, depending on the type of grouting method. Interconnected and overlapping columns can also be constructed in continuous rows in a primary/secondary sequence.

2.7.3.2 Grout Quantities

Jet grout quantities are less dependent on soil conditions than other types of grouting, and, therefore, the quantities reflect the design requirement, i.e., for underpinning design, the treated quantity and quality requirements depend upon the load imposed and the ultimate bearing capacity which can be achieved by the in-situ soil conditions.

2.7.3.3 Grout Types and Selection

For jet grouting, the grout typically consists of Portland cement with water/cement ratios of 0.8 to 1.2, although values as low as 0.5 have been used in three-fluid applications. Bentonite and other additives may be used depending on the specific project, but are relatively rare. Increasing use is being made of slag cements, which are typically 50% slag and 50% Portland cement.

2.7.4 Cost

Jet grouting is designed to solve problems in the ground that are normally untreatable by other ground modification methods. The cost of jet grouting can vary greatly, depending on the complexity of the project and the depth of treatment. Costs on complex projects in clay such as the Boston Central Artery project were approximately $150 per cubic yard of ground treated (in 1994). The typical cost currently varies from $115 to $230 per cubic yard.

Table 8-7 presents jet grouting costs for underpinning and excavation support and seepage applications, based on evaluation of more than 65 projects completed in the United States.

Table 8-7. Range of Jet Grouting Prices

Description	Unlimited Headroom (< 36 feet)	Restricted Headroom (10 feet to 13 feet)
Underpinning and excavation support 3.0 to 3.6 feet diameter per/yard of depth	$95 – $550	$490 – $650
Seepage control 3.0 to 3.6 ft. diameter per/yard of depth	$30 – $115	$30 – $200

The costs shown include mobilization, testing, and demobilization, which ranged from $25,000 – $50,000. These items are project specific and will vary depending on project size, but typically would represent 5% – 15% of overall costs. These costs indicate a large variation and, in general, are for projects smaller than the Central Artery in Boston. Jet grouting may also be measured as 1) mobilization, demobilization, and testing as a lump sum or 2) as a seepage barrier wall or underpinning project measured per square yard.

The items associated with the cost of jet grouting and approximate cost ranges are shown in Table 8-8. Cost ranges are based on a variety of projects from 2004 through 2008.

Table 8-8. Jet Grouting Project Costs 2004 to 2008 – Unit Prices and Factors Affecting Cost

Pay Item Description	Quantity Range	Unit	Low Unit Price	High Unit Price	Factors Which May Potentially Impact Costs
Mobilization	1	LUMP SUM	$25,000	$150,000	Equipment mobilized includes: drill rig(s), compressor(s), grout mixers, and pumps. Mobilization cost increases for distances greater than 500 miles. Phased projects may require multiple mobilizations.
Jet Grouting	Greater than 500	CY	$100	$750	Grout cost is sensitive to the grout mixture proportions, particularly the quantity of Portland cement required per cubic yard. Unit costs are far higher for locations which have headroom constraints.

Payment for jet grouting typically consists of a grouting pay item measured per cubic yard (CY). The associated additional costs included in the bid item are:

- Layout of a grouting pattern
- Disposal of spoils
- Instrumentation, monitoring, and quality control

Other costs associated with jet grouting which are measured and paid for separately include mobilization.

Project characteristics and constraints should be closely examined to determine the degree to which any of these factors may influence the actual cost associated with jet grouting. Note that a pre-production test program is essential and must be paid for. This can range widely depending on the scope, complexity, instrumentation, etc.

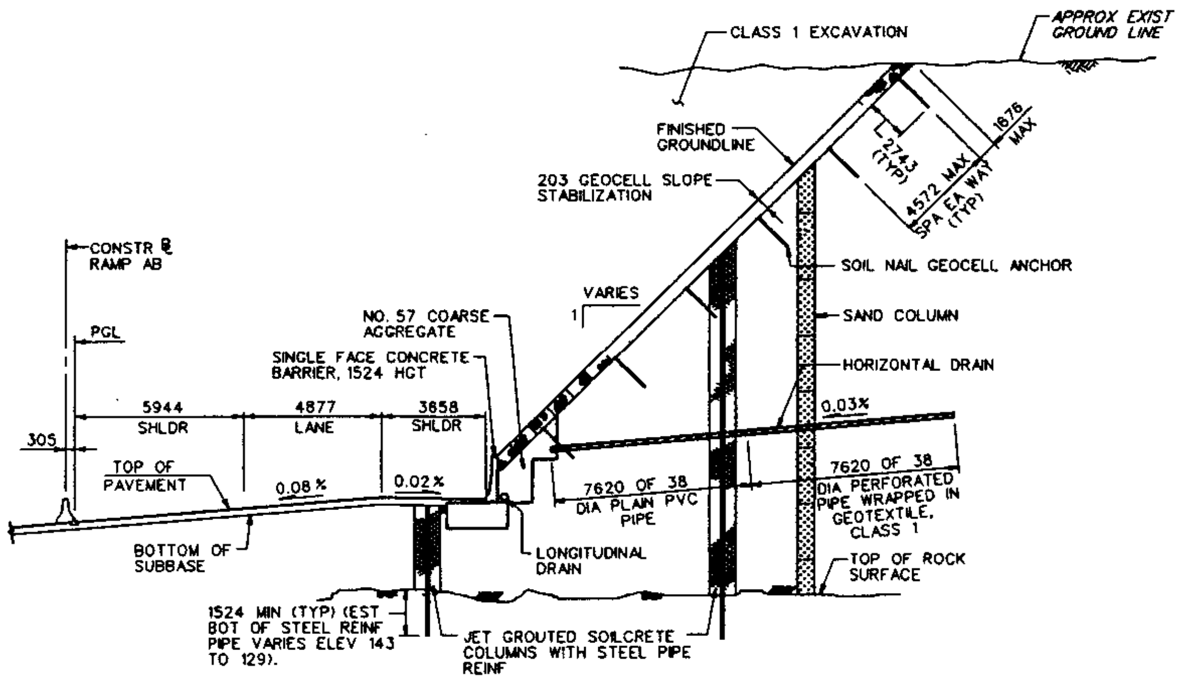
2.7.5 Case History

2.7.5.1 Case History 1: Stabilization of Excavation of Deep Cut Sections Using Jet Grouting, Interstate 78 to Route 33 Ramp Construction, Pennsylvania Department of Transportation (Meyers et al. 2003)

The Pennsylvania Department of Transportation (PennDOT) constructed ramps to direct traffic between I-78 and Route 33 in eastern Pennsylvania where the pavement surface was below the ground level. Design constraints required a very steep slope (1:1) to avoid encroachment of the soil slope into an adjacent historical property. The final section was cut to a depth of 58.5 feet with the slope varying from 2:1 to 1.1:1 to 2:1 within a length of 105 feet. The slope variation was designed to meet the PennDOT stability requirement that specify a factor of safety of 1.5 for permanent slopes. Initially, several retaining wall system configurations were evaluated which did not yield feasible solutions to the soil slope stability problem. Reinforced soil-cement columns forming “cut-off” walls were constructed using jet grouting and the finished slope face was further stabilized using a “geo-cell” product that was pinned down with drilled soil anchors.

The soil at the project site was a carbonate rock formation consisting of limestone and dolomite. The bedrock formation is highly faulted, folded, and fractured due to its being prone to solution weathering with possibility of sinkhole occurrence.

The final slope variation was determined from a detailed slope stability analysis, which showed that use of jet-grouted columns reinforced using steel pipes increased the factor of safety from 1.2 to the minimum requirement of 1.5. Columns having a diameter of 2.5 feet were placed in a zigzag pattern in two rows – one near the toe of the slope and at a distance of one-third of the slope length from the toe, as shown in Figure 8-21. Soil nails consisting of 4 inch neat cement grout reinforced by 1 inch diameter steel bars were constructed on a 15 ft × 15 ft grid along the entire surface of the slope.



Meyers et al. 2003

Figure 8-21. Typical section of stabilized slope – PennDOT jet grouting project.

The construction specifications detailed a three-stage construction of soil-cement columns using a contractor-patented method to install columns and reinforcement simultaneously. Unconfined compressive strength testing resulted in acceptable strength values (approximately greater than 500 psi) when sulfur caps were used instead of neoprene caps during testing. The final cost of construction was $1,300,000 for the entire slope.

In this project, jet grouting was found to be an effective method to improve the factor of safety for slope stabilization. Jet grouting was also found to be highly cost-effective and required less time to complete, compared to other alternatives such as retaining walls.

2.8 Rock Fissure Grouting

Rock fissure grouting is a grouting technique most commonly used in dam and tunnel construction and rehabilitation for structural stability and groundwater control. Rock fissure grouting is primarily used to provide hydraulic cut-off zones of relatively low permeability, but it can also be used to bind together rock masses mechanically to enhance load bearing properties. The technique can also be applied on any project where there is a hydraulic or structural requirement to fill the fissures in a rock mass. For transportation facilities, potential applications include shaft repair and the remediation of deteriorating road or railway tunnels, and the stabilization of rock slopes. It can also be used as remedial grout curtains to prevent sinkholes and surface depressions caused due to dewatering and/or movement of fines in highways adjacent to active mineral quarries.

Similar drilling and grouting techniques are also widely used to locate and seal major voids in rock masses. These voids may be naturally created (e.g., karstic limestone features, or salt solution cavities) or can be due to human activities (e.g., mineral workings, such as coal or iron mines). Such voids can generate surface settlements and/or can permit the relatively easy flow of large volumes of water under hydraulic gradients.

Rock grouting with particulate materials normally falls into one of the following categories:

- **Curtain grouting** is the drilling and grouting of two or more lines of grout holes to produce a barrier to seepage. The curtain usually extends into materials judged acceptably impermeable.
- **Area grouting** (also known as "blanket" or "consolidation" grouting) normally consists of grouting a shallow zone in a particular area, utilizing grout holes arranged in a pattern or grid. Its purpose is to mechanically improve fractured and jointed rock. Deeper area grouting is sometimes performed in specific geologic conditions, such as fault zones, or to consolidate subsurface materials at shaft or buried structure locations.
- **Tunnel grouting** may be used to fill voids behind tunnel liners (contact grouting), treatment of material surrounding the bore, or for seepage control. Pre-excavation grouting from the surface or from the face may be required for ground strengthening and water control on some tunnel projects.
- **Backfilling** of subsurface exploration boreholes and grout holes is important to maximize structural stability, to control water, or to prevent passage of contaminants to underlying strata. This may also be performed for soil borings.

2.8.1 Applications

As mentioned above, the most common use of rock grouting today is in dam and tunnel construction and rehabilitation, especially for structural stability and groundwater control (Weaver and Bruce 2007). For transportation facilities, potential applications include shaft repair and the remediation of deteriorating road or railway tunnels, and the stabilization of rock slopes.

2.8.2 Advantages and Potential Disadvantages

The advantages and disadvantages of rock fissure grouting are as follows.

2.8.2.1 Advantages

Rock fissure grouting can be used to repair weak or permeable rock masses where limited alternatives exist such as costly removal and replacement to abandoning the site. The main advantages of the technology are (Lombardi 2003):

- Reducing the permeability of the rock mass
- Reducing the deformability of the rock mass
- Increasing the strength of the rock mass particularly against shear forces

2.8.2.2 Potential Disadvantages

Inefficiencies in rock fissure grouting will occur due to poor design and poor field practices, which include:

- Inducing uplift and damage to foundations, resulting from excessive pressures.
- Premature plugging of fissures by thickening the mix too quickly, by unsuitable injection methods or formulations, or by using inappropriate drilling and flushing techniques.
- Improper hole spacing or orientation of grout holes.
- Inappropriate verification.

These disadvantages can be rectified by utilizing knowledgeable and experienced personnel to design, construct, supervise, inspect, and control the drilling and grouting operations.

2.8.3 Feasibility Evaluations

Factors affecting geotechnical, environmental, and site-related feasibility of rock fissure grouting are described in this section.

2.8.3.1 Geotechnical

The main consideration for the use of rock grouting to seal cracks and fissures, or injecting grout for either water control or structural improvement purposes, is the grain size of the particulate grout compared to the width of the rock fracture to be grouted. The groutability ratio N_R for rocks fissures is

$$N_R = \frac{\text{Width of fissure}}{(D_{95})_{\text{GROUT}}} \qquad \text{[Eq. 8-6]}$$

Where “D” is defined as the grout diameter, and the subscript is the percent finer (Mitchell 1981). As the rock characteristics cannot be changed, the fineness of the grout should be controlled and its rheological properties carefully engineered so that the N_R number for rock grouting feasibility is nearer 3 than 5, as follows:

- $N_R > 5$: Grouting consistently possible
- $N_R < 2$: Grouting not possible

It is important to measure the in-situ permeability of the rock mass in advance, since this fundamentally influences the design, construction, and verification processes. Stability of rock mass is also vital for grouting design. Lugeon value is more convenient for use in designing and constructing the grout curtain (Weaver and Bruce 2007), and is defined as the permeability of grout in the rock mass and is equal to 0.11 inch^2/psi (1 L/m/bar) at a test pressure of 145 psi (10 bar).

1 Lugeon (Lu) = 0.5×10^{-6} inches/second (1.3×10^{-5} cm/s)

Testing at different test pressures in an up-and-down order (low-moderate-high-moderate-low) is useful to study the elasticity of rock fissure opening, grout flow characteristics, presence of voids in the rock mass (Houlsby 1990, Weaver and Bruce 2007) and occurrence of hydrofracture (Littlejohn 1992).

2.8.3.2 Environmental

Care must be exercised when performing grouting in rock where the grout could leak into a body of water. The depletion of oxygen by the grout or the effect on the pH of the water could lead to a fish kill.

2.8.3.3 Project Conditions

Before deciding if grouting is appropriate for a particular site, a thorough subsurface investigation should be conducted. Rock masses can be highly variable, including weak or loose rock, rock with stress fractures, rock with large voids, and rock with open fractures and/or possessing high permeability. Some rock masses may be erodible or soluble. Permeability testing (in situ, Lugeon value) and the use of an optical televiewer are essential components in any such investigation.

Often a design phase test program is warranted to determine the effectiveness of a rock grouting program. Based on the data obtained from this program, a final grouting design, and the associated program cost estimate, can then be logically developed. The site-specific explanation should be tied into knowledge of the local geology.

2.8.4 Limitations

Over the years, experience has shown that it can be difficult to pre-assess the cost of a rock grouting program. Site geology can be extremely complex, with widely differing subsurface conditions existing within the site boundaries. Even when a test program is performed, the statistical results may still not be sufficient to determine project costs with a reliable degree of accuracy. These factors prevent accurate cost estimation of rock grouting operations. However, recent grout curtains in carbonate terrains have costs between $25 and $80 per sq. feet of curtain.

2.8.5 Design Considerations

The design of a grouting program consists of defining the areal extent of grouting, the number of rows of grout holes required, determining the appropriate grout materials, initial hole spacing, inclination, and diameter, quantities of grout, grouting equipment methods, and parameters, developing performance and verification requirements, determining the required monitoring tools, and developing contract documents (plans and specifications). The precise goal of the program must be clearly stated. This may be a specific residual permeability as measured by post-grouting tests, or an increase in rock mass strength or homogeneity, as illustrated by core-sample testing, load testing, or cross-hole seismic methods.

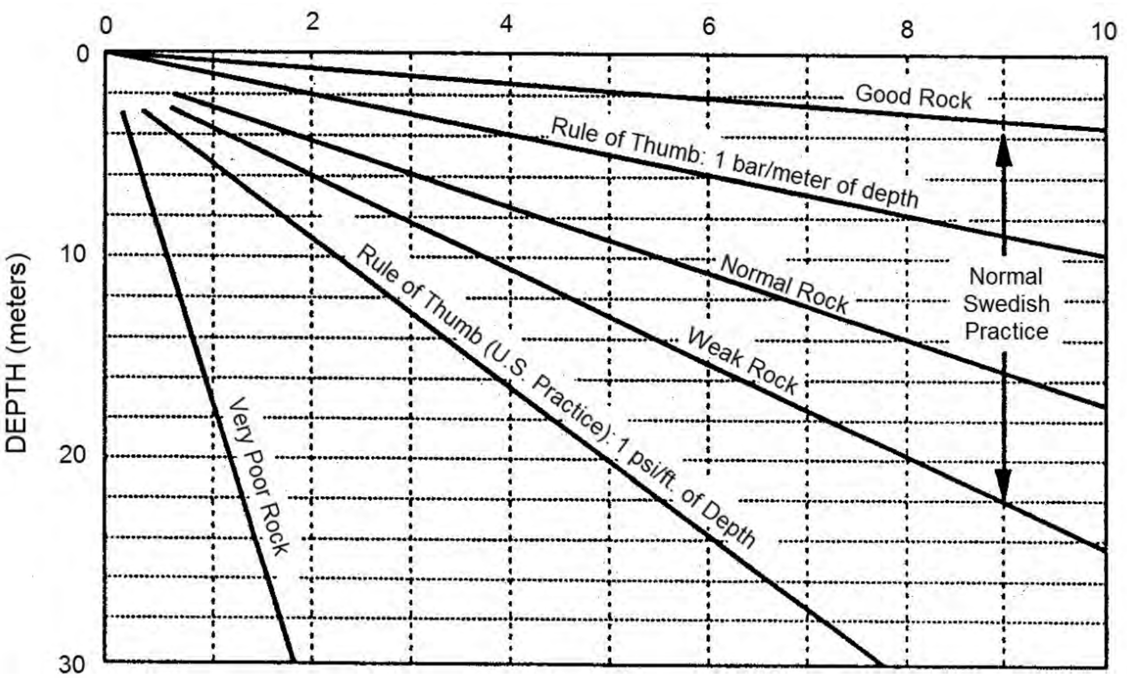
2.8.5.1 Grout Types and Selection

Rock fissure grouting is primarily done with particulate grouts. The exact mix formulation must reflect the fluid and set properties that are required to enhance penetrability, and to provide a durable product (Bruce et al. 1998). Whereas traditional practice was incorrectly based on neat cement grouts, current practice features the use of suites of multi-components, balanced formulations with carefully controlled fluid and set properties (ASCE 2003). The physical and engineering properties, fluid and setting characteristics, and constituents of cement-based grouts have been described in detail by Weaver and Bruce (2007).

2.8.5.2 Design Procedure/Program and Considerations

Major components of the subsurface investigation for rock grouting include leakage potential, areal and structural geology, in-situ stress conditions, hydrogeology, geochemistry, and compatibility of in-situ and grouting materials. Rock mass discontinuities, especially frequency and aperture, are vital to record, as is the in-situ permeability of the rock mass. The presence and characteristics of anomalous conditions are ascertained, and appropriate treatment planned.

Drilling and flushing methods are usually selected by the contractor, although the use of water flush is essential for fissure grouting. The drilling equipment is further used to remove by washing or flushing, all drill cuttings and turbidity from the grout hole. After flushing, pressure washing is sometimes performed using the pressure testing equipment. Pressure washing and pressure testing are conducted immediately before pressure grouting operations are commenced. Pressures used for pressure washing and testing should not exceed the maximum allowable grouting pressures and, indeed, should be used to determine the latter. Washing continues until clay or washable materials are removed from an interconnected hole or surface leak, or as long as the rate of water injection increases at a given pressure. A clay dispersant can also be used. A pressure test using clean water is often performed following pressure washing, either at a constant pressure or at multiple pressures (Houlsby 1990). Regarding grouting pressures, there are various "rules of thumb" ranging from 1 to 4 times the theoretical weight of rock above the injection point, as summarized in Figure 8-22.



Units: 1 bar = 14.5 psi, 1 m = 3.28 feet or 40 inches

Weaver and Bruce 2007

Figure 8-22. Rock grouting injection pressures used in Swedish grouting practice.

Many factors will dictate the site-specific choice, such as geological and structural conditions; but the maximum safe pressure must be confirmed in preconstruction testing.

There are three basic methods used for grouting stable rock masses:

- Upstage (ascending stage)

- Downstage (descending stage) with top hole packer
- Downstage with down hole packer

Upstage grouting and downstage grouting are shown in Figure 8-23 (left) and Figure 8-23 (right), respectively.

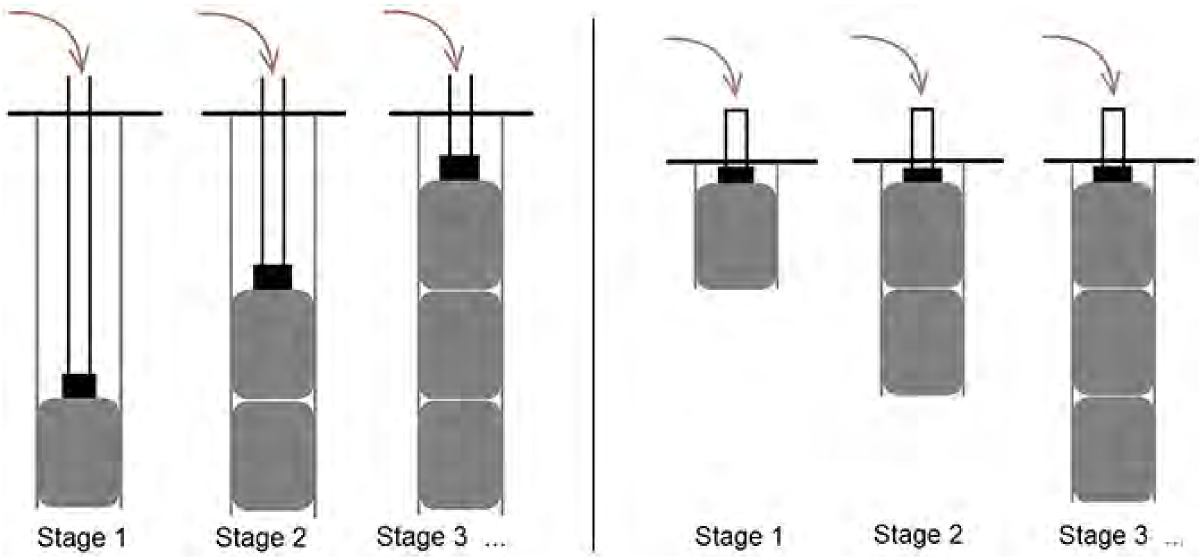


Figure 8-23. Rock grouting techniques: upstage (left) and downstage (right).

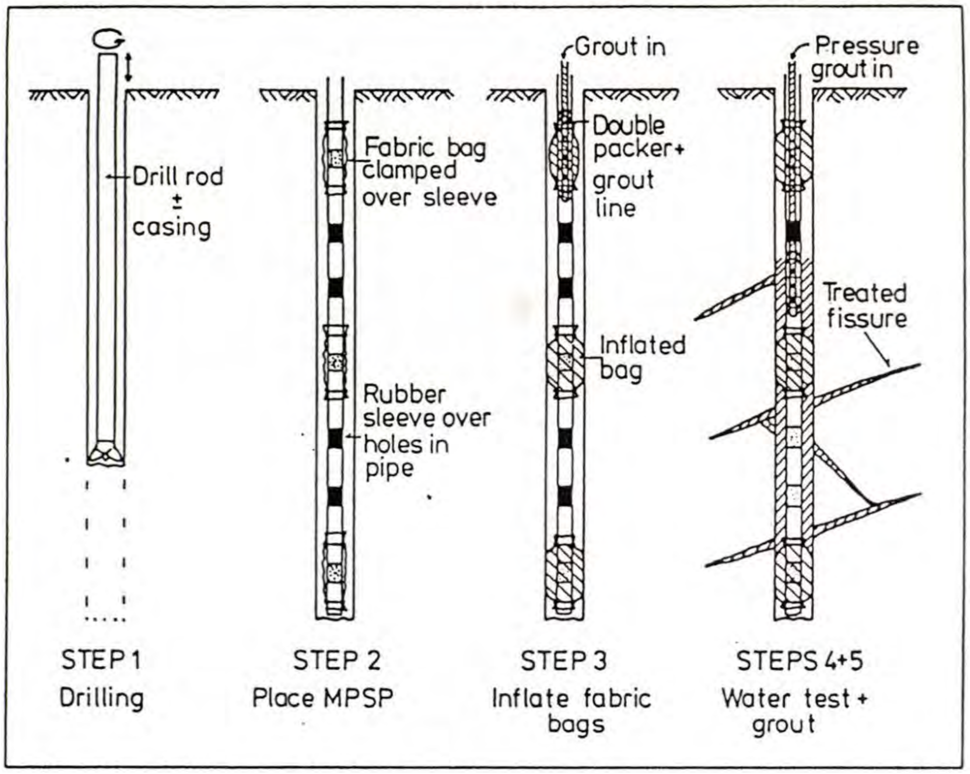
The advantages and disadvantages of upstage and downstage methods are summarized in Table 8-9.

Table 8-9. Major Advantages and Disadvantages of Downstage and Upstage Grouting of Rock Masses

	Downstage	Upstage
Advantages	• Ground is consolidated from top down, aiding hole stability, packer seating, and allowing successively higher pressures to be used with depth without fear of surface leakage. • Depth of hole need not be pre-determined: grout take analyses may dictate changes from foreseen, and shortening or lengthening of hole can be easily accommodated. • Stage length can be adapted to conditions as encountered to allow "special" treatment.	• Drilling in one pass. • Grouting in one repetitive operation without significant delays. • Less wasteful of materials. • Permits materials to be varied readily. • Easier to control and program. • Stage length can be varied to treat "special" zones. • Often cheaper, since net drilling output rate is higher.
Disadvantages	• Requires repeated moving of drilling rig and redrilling of set grout: therefore, process is discontinuous and may be more time consuming. • Relatively wasteful of materials, and so generally restricted to cement-based grouts. • May lead to significant hole deviation. • Collapsing strata will prevent effective grouting of whole stage, unless circuit grouting method can be deployed. • Weathered and/or highly variable strata problematical. • Packer may be difficult to seat in such conditions.	• Grouted depth predetermined. • Hole may collapse before packer introduced or after grouting starts, leading to stuck packers and incomplete treatment. • Grout may escape upwards into (nongrouted) upper layers or the overlying dam, either by hydrofracture or bypassing packer. Smaller fissures may not then be treated efficiently at depth. • Artesian conditions may pose problems. • Weathered and/or highly variable strata problematical.

The competent rock available on most dam sites is well suited for upstage grouting, and this has historically been the most common method. Downstage methods have recently had more demand reflecting the challenges and difficulties posed by more difficult site and geological conditions at remedial and hazardous waste sites. It is not unusual to find that the uppermost stage (in typically the poorest rock) must be downstaged, but that the other stages can be upstaged. In some cases of extremely weathered and/or collapsing ground conditions, even

descending stage methods can prove impractical, and the MPSP (Multiple Packer Sleeve Pipe) shown in Figure 8-24 is used as the preferred alternative (Bruce and Gallavresi 1988).



Bruce and Gallavresi 1988

Figure 8-24. Multiple packer sleeve pipe (MPSP) process.

2.8.5.3 Performance Monitoring

Detailed performance monitoring and evaluation is an integral part of the grouting program. Real-time evaluation of records of drilling, pressure testing, and grouting operations enables any necessary technical changes to be made as the project progresses; hence real time computer monitoring/recording should always be mandatory (see Bruce 2012). For example, the geologic profile that is developed from test boring data and upon which the design of the rock grouting program is based, may not accurately reflect the subsurface conditions overall, since the number of exploratory test borings made on a project is limited by cost considerations. During the drilling process, deviations from the anticipated rate of progress and rock or mud cuttings recovered are indicators of an unexpected subsurface condition. This information serves to "fill in the gaps" between test borings, allowing a more detailed geologic profile to be developed. All of this information is included in the as-built report.

Computerized monitoring, recording, and analysis of grouting operations provides instantaneous, accurate information on progress at any given location. This allows immediate

input to the field construction crews as to progress and necessary changes. First used in the United States in 1983 by the Bureau of Reclamation at Ridgeway Dam in Colorado, computerized grout monitoring was highly successful, and is now standard practice as a monitoring and control mechanism in North America (ASCE 2003, ASCE 2012, Weaver and Bruce 2007, Davidson 1984, Bruce et al. 1998).

The performance of the grouted rock mass must also be monitored with time. For example, if the goal is water tightness, seepage flows and pressures should be monitored during service. For blanket grouting, structural movements should be monitored, and so on.

2.8.6 Cost Data

2.8.6.1 Bidding Methods

Rock grouting may be performed as part of a general construction contract or under a separate contract. For rock grouting, as for all other grouting, pay items for individual aspects of the work are listed separately. This approach, while not common in general construction, is usual for grouting and is the approach of choice of government agencies, based on experience. Costs for routine instrumentation, though specified, is typically included within other items.

Because of uncertainties involved in rock grouting (i.e., the requirement for maximum flexibility to meet field conditions and the exploratory nature of grout programs), accurate estimates of quantities are extremely difficult. Many contracts contain language that reserves the right to increase or to eliminate any part of the drilling and grouting program without changing unit prices.

Grout and exploratory hole drilling are paid on the basis of the linear feet of holes actually drilled, typically including the cost of washing. In most contracts, pressure testing and washing are separate, hourly-based pay items, because the inspecting agency on site might direct the time that these procedures are to continue. Re-drilling set grout is typically priced at 50% the rate for rock drilling. Materials are paid on the basis of weight of each component injected into the grout holes. Grout injection (or placement) is paid by the pump hour.

2.8.6.2 Cost Estimation Methods

The volume and extent of work involved in a drilling and grouting program can only be approximated in advance of construction. Quantities are estimated for bidding purposes, but substantial variations are common especially in the grouting items. The contract specifications and bid items should be prepared so that the estimated quantities for each of the bid items may vary substantially without affecting unit prices. However, a concerted

effort must be made to estimate the quantities of drilling and of grouting materials (e.g., grout take) that will be required. Pre-construction test-grouting programs, boring evaluations, past experiences and unit-take estimates are frequently used for estimating purposes.

The contract drawings and specifications should clearly indicate the drill hole spacing, sequencing, direction, maximum angle, maximum depths, and allowable deviation there from. These requirements can be used as the basis for refinement as the grouting program is implemented during construction. The amount of drilling should be estimated on the basis of the project as planned and shown on the drawings, and the amount of drilling anticipated for each drilling item should be shown. The related quantities of water testing, grouting, materials, and so on should also be carefully spelled out.

The following additional items should also be included in an estimate or bid schedule:

- Drilling Exploratory and Verification Holes – To determine the effectiveness of the grouting or portions thereof during grouting operations, it will be necessary to drill such holes at key locations. Drilling of exploratory and verification holes will be measured for payment on the basis of linear feet of holes actually drilled.
- Drilling Drain Holes – The drilling of drain holes should be covered by separate items for each hole size. Should both drilling in the open and from galleries be required on the same project, separate items for these conditions may be desired. The spacing and the depth of drain holes can ordinarily be predetermined with a greater degree of accuracy than can grout holes. The quantity for each item should be expressed in linear feet.
- Instrumentation – This included all instrumentation other than that integral to control or analyze the drilling and grouting data. (The latter data systems can also be priced separately, either as a lump sum or by instrument). Monitoring of instruments may be a separate item.

The type of rock to be treated and the purpose and performance objective of the grouting program are major factors affecting the cost of any rock grouting project. When preparing a bid package, it is recommended that input be sought from local federal, state, and private organizations, as well as from specialty contractors. As a general guide, it may be estimated that a grout curtain may cost $25 to $80 per square yard of curtain, including all drilling and grouting activities and materials.

2.8.7 Case Histories

Two case histories are presented to describe the use of grout curtains in dams, which is the most important application of rock grouting.

2.8.7.1 Case History 1: State of the Art in Computer Monitoring and Analysis of Grouting – Penn Forest Dam, Pennsylvania; Patoka Lake Seepage Remediation Project, Indiana, and Hunting Run Dam Project, Virginia (Dreese et al. 2003)

Dreese et al. (2003) describe the technical and economic benefits of using computer monitoring and real-time data analysis in grouting application using three case histories. The advantages of computerized data collection can be summarized as higher frequency (more data), higher grouting pressures, faster and consistent grouting operations, and better allocation of manpower and resources. Plots of lugeon value or flow rate divided by effective injection pressure versus time are extremely useful to identify problems such as unsafe grouting pressures.

Monitoring of grouting was categorized into three levels depending on its applicability and use:

Level 1: Dipstick and Gage. This level of monitoring was used prior to 2000 and is almost no longer used in modern grouting practice. A dipstick is used to measure grout take, a pressure gage to measure water or grout injection pressures and a water meter to measure water intake (Wilson and Dreese 1998). Frequency of data collection is 5 to 15 minutes for obtaining stable readings, and plots of average grout take per time interval are plotted manually.

Level 2: Real-Time Data Collection, Display, and Storage. In this system, real time data measurements of flow and pressure are collected by electronic devices and are automatically recorded and displayed on other devices. This level of monitoring allows engineers to make analyze displayed trends of flow, pressure, and other selected parameters. However, patterns or anomalies cannot be easily identified by onsite personnel from the large amount of data collected.

Example: Computer Aided Grout Evaluation System (CAGES)

Level 3: Advance Integrated Analytical (AIA) Systems. AIA systems are far more advanced than Level 2 systems in terms of integrating data collection, real-time data display, analytical and query capabilities, and CAD. IntelliGrout is an AIA system capable of graphically displaying real-time data of geological features and stratigraphy, hole geometry, and grout and water test data, in conjunction with CAD. This helps to quickly identify patterns, anomalies, deviations, and special areas of interest. Level 3 systems are recommended to be used for projects whose overall cost exceeds $750,000 whereas Level 2 systems may be used where project cost is more than $250,000.

Example: IntelliGrout System, Advanced Construction Techniques, Ltd. and Gannett Fleming, Inc.

The Penn Forest Dam in Pennsylvania shown in Figure 8-25 was constructed to supply water to the city of Bethlehem, Pennsylvania.

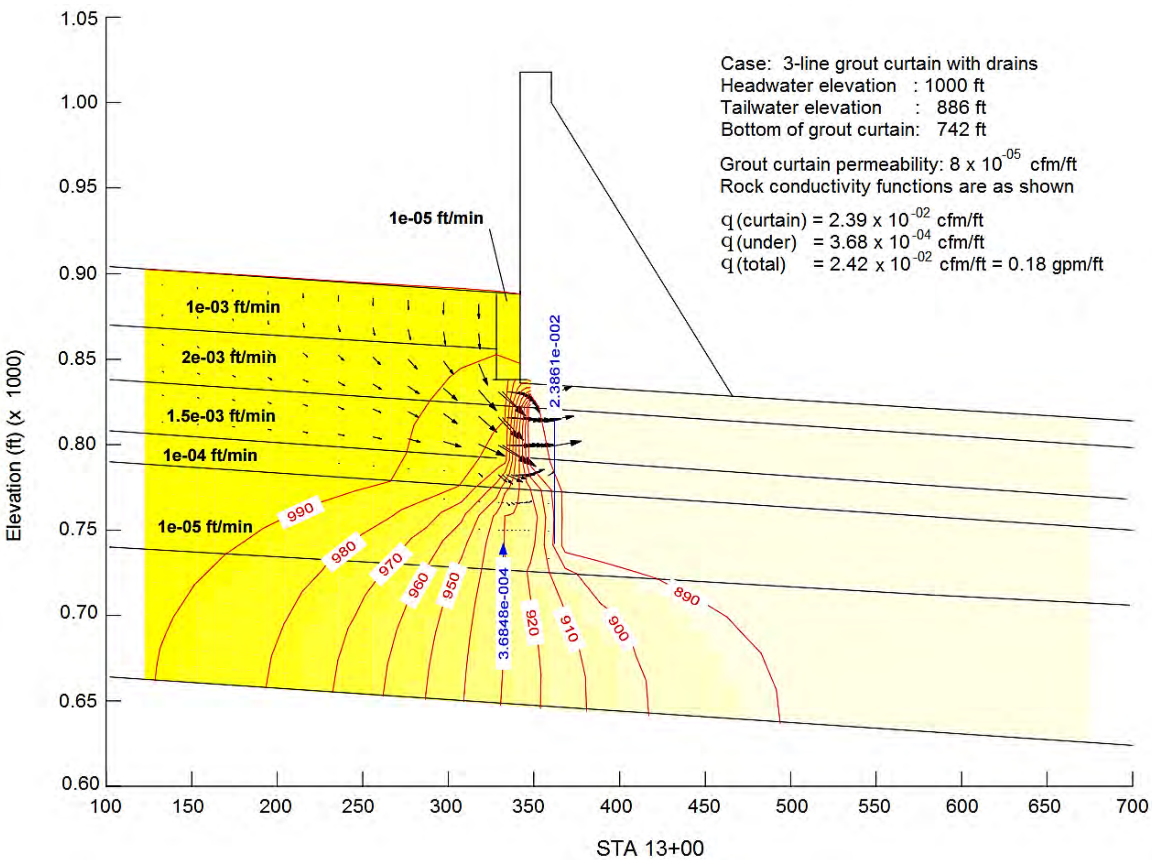
Dreese et al. 2003

Figure 8-25. Penn Forest Dam, Pennsylvania.

The dam was approximately 180 feet high and 2,000 feet long, and included a triple-row grout curtain. The first row (Line A) was constructed using neat, conventional cement-based grouts and Level 1 monitoring. The second and third rows (Lines B and C) were constructed using more balanced and stable modified cement-based grouts using a Level 2 system. The additives used to modify the grout for Lines B and C were bentonite, fly ash, Welan gum, and superplasticizer. The differences in the properties of the modified, stable grout and the neat grout are as follows:

- Slightly higher viscosity due to additives
- Lower cohesion due to deflocculating effect of superplasticizer
- Lower bleed water accumulation
- Lower pressure filtration coefficient, indicating greater pumping distances without caking
- Lower overall compressive strength, but sufficient for grouting application (greater than 200 psi)

CAGES was used to perform three major functions – continuous graphical monitoring of lugeon value, evaluating suitability of initial grout mix and grout takes, and displaying additional data such as grouting time, spread radius, and effective grouting pressure. The introduction of advanced stable grouting materials and implementation of electronic monitoring and computer-aided analysis for the Penn Forest Dam showed an improved grouting quality at a reduced overall cost. Figure 8-26 shows the reduced flows through the dam after grouting.



Dreese et al. 2003

Figure 8-26. Final grout curtain – permeability reduction in valley section.

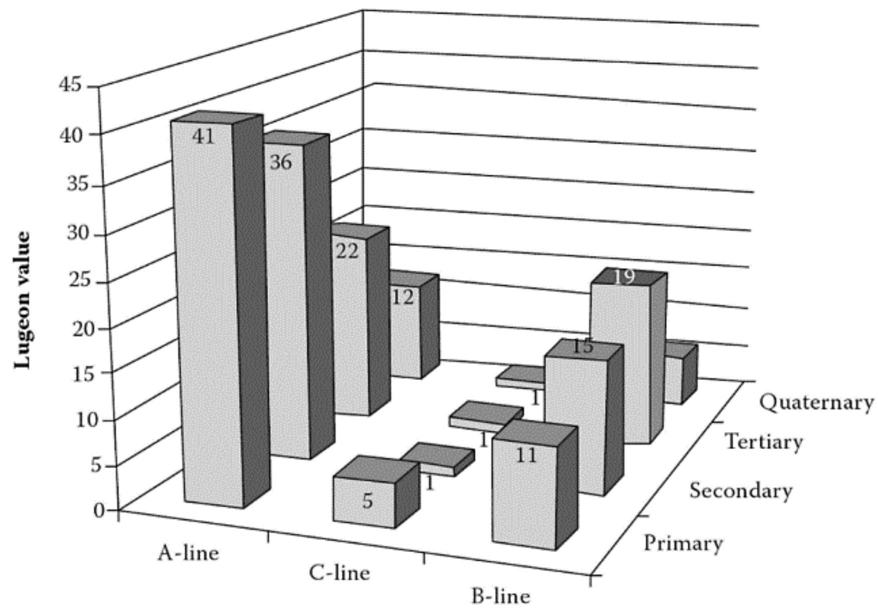
The Patoka Lake Seepage Remediation Project in Indiana involved grouting of a limestone ridge between the left abutment of the dam and the emergency spillway as shown in Figure 8-27.

Courtesy Advanced Construction Techniques, Ltd.

Figure 8-27. Patoka Lake Dam grouting.

The project was completed by Louisville district US Army Corps of Engineers (USACE) and a contractor-engineer team using balanced, stable grouts and computer monitoring. The contractor was selected on Best Value Selection basis rather than the traditional low bid, and the project was overseen by a full-time USACE geologist.

Advantages of using Level 2 technology for this project were reduced operational time and inspection costs, ability to confidently use higher pressures, and generation of superior contract records and documentation. The grouting resulted in a decrease in permeability in areas surrounding the dam by up to three orders of magnitude. The average residual Lugeon value of the grouted zone (C-line) was approximately 1 Lugeon, as shown in Figure 8-28.



Dreese et al. 2003, Courtesy Gannett Fleming, Inc.

Figure 8-28. Lugeon values – Patoka Lake Dam post-grouting.

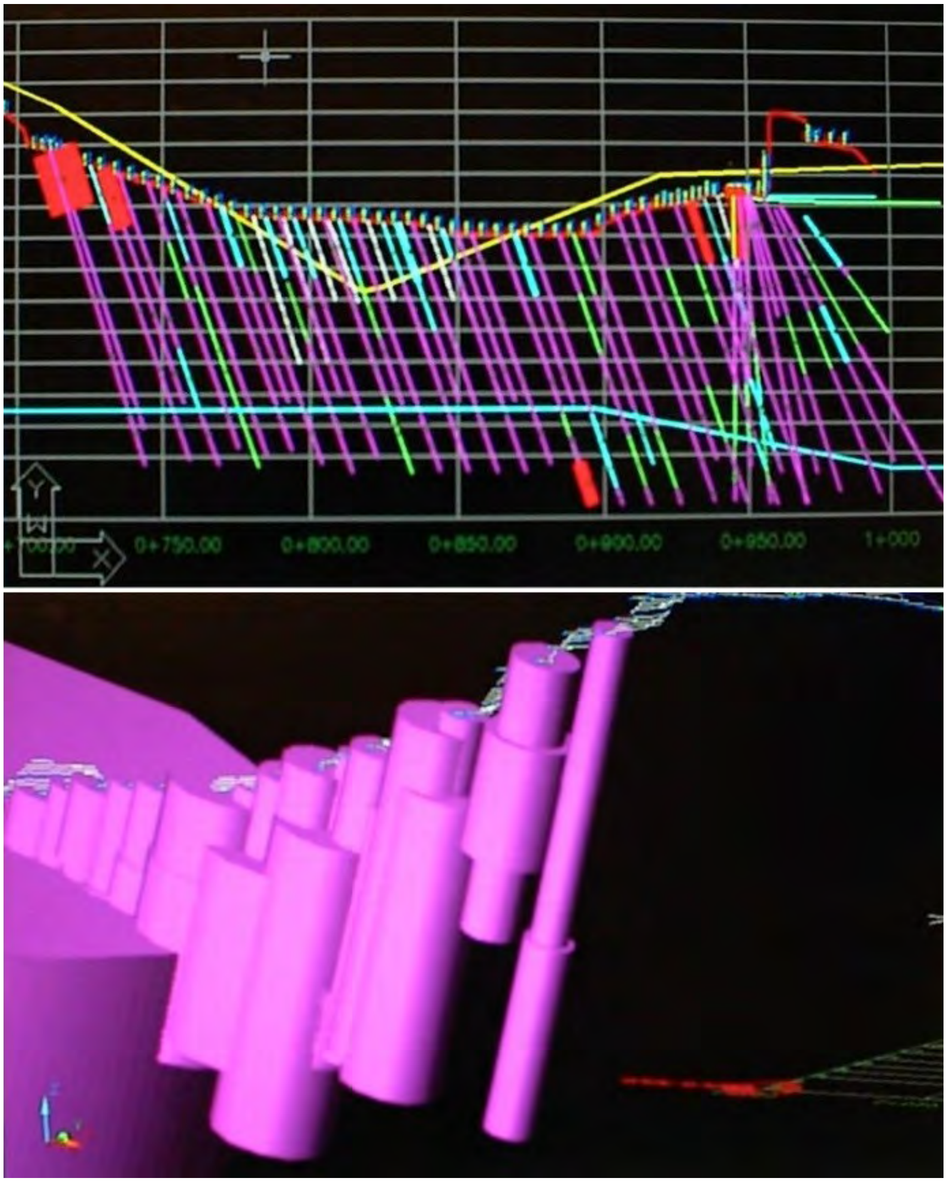
From verification testing, it was determined that the grouting zone could withstand pressures in excess of the expected hydraulic heads without hydro-fracturing through soil seams within the grouted limestone mass. It was concluded that balanced, stable grouts and computer monitoring are a technically- as well as cost-effective alternative to concrete cut-off wall methods to reduce permeability.

Grout curtains in the Hunting Run Dam in Spotsylvania, Virginia were constructed using IntelliGrout systems to reduce the permeability to lower than a defined performance criterion of 5 Lugeons. The basic grout curtain was a single line, 1100 feet long curtain constructed up to a depth of 120 feet, with a design provision for additional curtain lines of variable depth as required to achieve the permeability criterion. The IntelliGrout system provided both 2D and 3D displays of water testing results, which facilitated the location and isolation of specific geological features that required additional treatment or other modifications. Figure 8-29 shows the grouting operations at Hunting Run Dam.

Courtesy Advance Construction Techniques, Ltd.

Figure 8-29. Grouting operations at Hunting Run Dam, Virginia.

The 2D and 3D display of water testing results, shown in Figure 8-30, shows the high permeability zone to the right of the conduit and dipping to the left, and the high permeability weathered zone near the center of the valley, which was identified from subsurface investigation.



Courtesy Advanced Construction Techniques, Ltd.

Figure 8-30. Hunting Run Dam grouting.

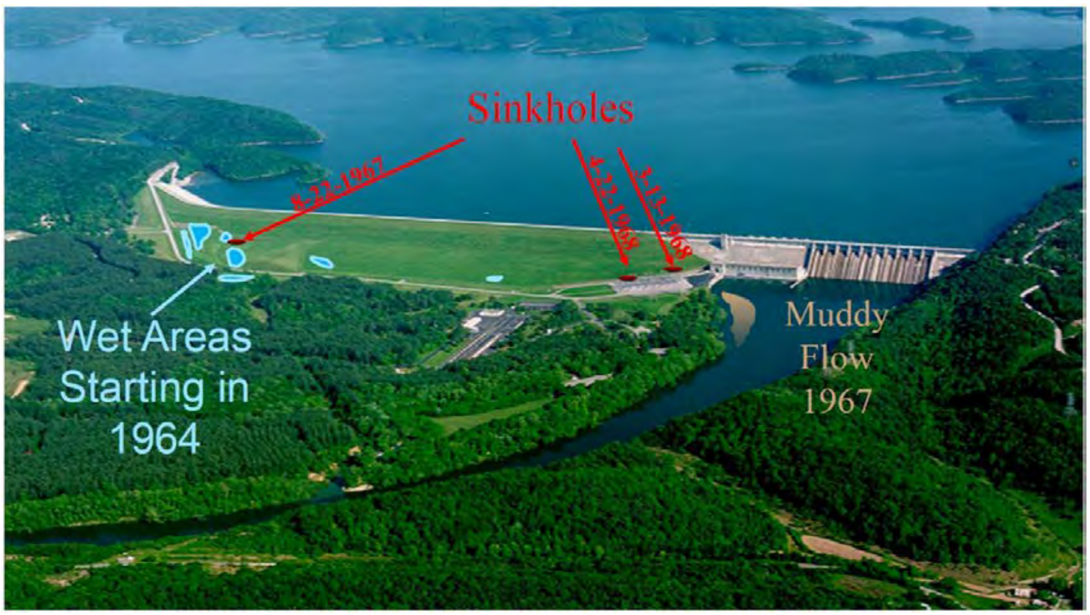
The high permeability feature under the conduit, which followed a weathered intrusive dike was unknown during design, and is highly unlikely to be noticed from conventional wall charts, resulting in concentrated residual leakage. The zone was quickly identified by the system operators, where the planned holes were either deepened or additional holes added to achieve the performance criterion.

The overall construction cost of the grout curtain was approximately $1.1 Million. The IntelliGrout system provided substantial value and economic advantages in terms of reduced inspection force, reduced time for peer review of grouting results, and better visualization of

geologic conditions and grouting results based on which changes were made to the grouting program to achieve the desired performance.

2.8.7.2 Case History 2: Wolf Creek Reservoir (Bruce et al. 2014)

Wolf Creek Dam built on Cumberland River in south central Kentucky is a 5,736 feet long and up to 258 feet high. The dam geology is exclusively karstic limestone, characterized by an extensive interconnected network of solution channels in the limestone foundation. Seepage problems in the reservoir arose due to formation of two sinkholes near the downstream toe of the embankment and muddy flow in 1968, which caused piping of filling materials and collapse of overburden and embankment into the voids. The location of sinkholes and muddy flow are shown in Figure 8-31.



Bruce et al. 2014, Photo source USACE

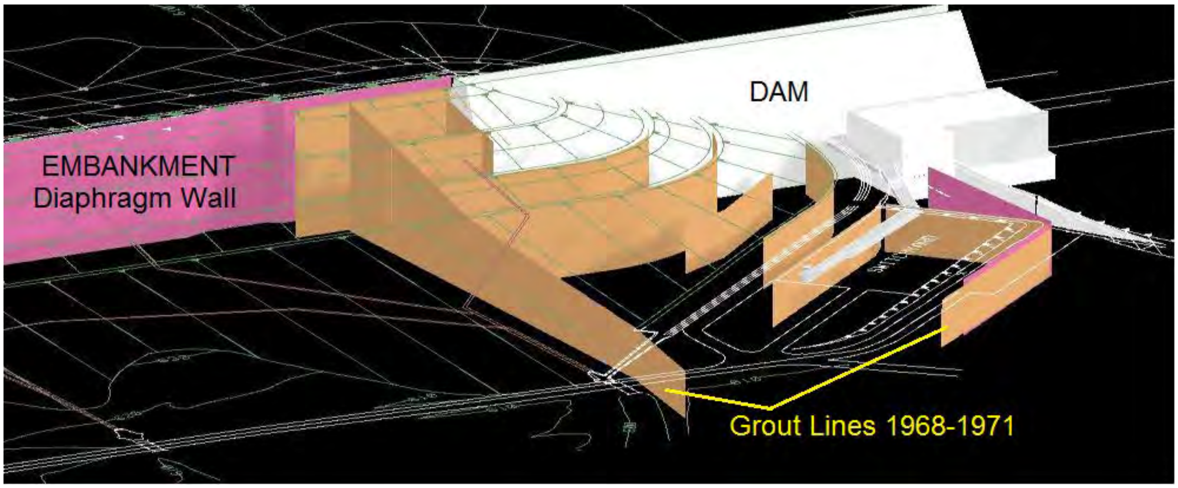
Figure 8-31. Wolf Creek Dam, south central Kentucky.

Grouting activities at the Wolf Creek Dam were performed in three phases as described below:

Phase 1 – 1942 to 1943. Six hundred grout holes were installed along the core trench length of 4,380 feet upstream of the embankment using downstage method and neat cement grouts with an average w/c ratio of 0.66 (Bruce et al. 2014). Holes were drilled generally on 10-foot centers and typical depth of 50 ft. for a total linear footage of 32,761 feet, with 112 holes deepened to accommodate geometry.

Phase 2 – 1968 to 1971 and 1973 to 1975. Emergency grouting was undertaken in the first part in 1968 to address rapidly deteriorating foundation conditions under the embankment

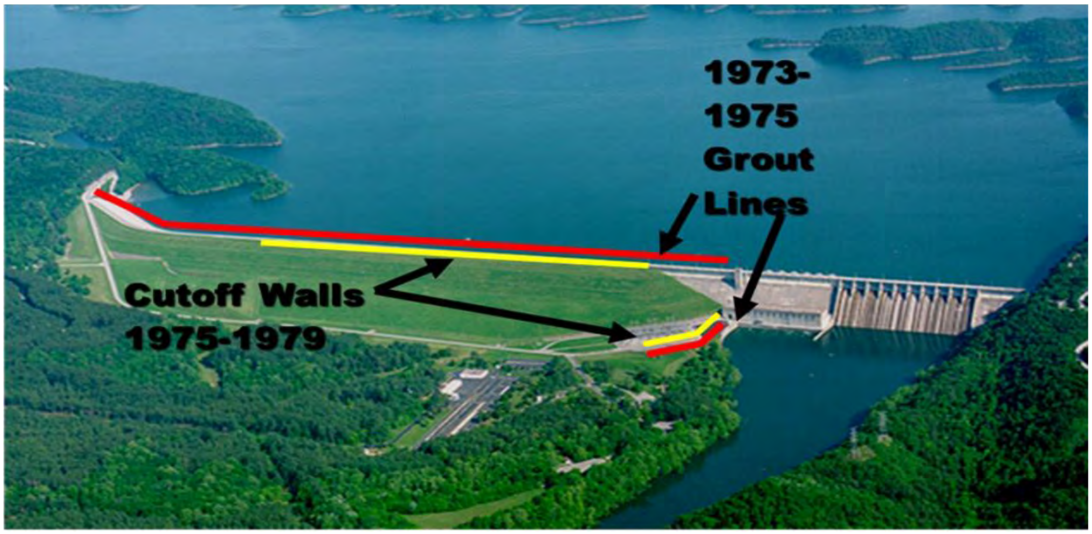
due to piping and sinkholes. Grout holes of diameter 6.75 inches at 2.5 feet center to center were drilled at a penetration rate limited to 0.6 feet per minute. The holes were filled using 1.5-inch diameter pipes with grout allowed to flow under gravity. Blockage between the injection pipe and casing was monitored by pressure gage at the surface and holes were filled until rejection of grout take. The grout lines constructed in Phase 2 are shown in Figure 8-32.



Bruce et al. 2014, Image source USACE

Figure 8-32. Wolf Creek Dam – grout curtain layout.

The second part from 1973 to 1975 consisted of exploratory drilling and grouting along the proposed concrete cutoff wall. A total of 852 holes were drilled along the total alignment of grout lines as shown in Figure 8-32. Two concrete diaphragm cutoff walls were installed in the period of 1975–79, one wall of variable depth and length 2,200 feet along the crest of the dam, and another wall of length 600 feet and depth primarily 95 feet along the downstream toe in the switchyard as shown in Figure 8-33.



Bruce et al. 2014, Photo source USACE

Figure 8-33. Wolf Creek Dam – cutoff walls and grout lines.

Phase 3 – 2007 to 2008 Interim Risk Reduction Measures. The primary goal of Phase 3 was to reduce seepage flow through the dam to reduce the risk of dam failure and to reduce the risk of slurry loss during cutoff wall construction. The major grouting operations performed in Phase 3 consisted of three sub phases – a two-line grout curtain of length 3,840 feet along the proposed concrete diaphragm wall, a 200 feet long single-line foundation grout curtain drilled from the east end gallery to concrete monolith of dam, and foundation exploratory hole and instrumentation along the core trench and dam embankment.

All drilling and grouting operations were performed from a concrete platform, constructed along the entire upstream slope of the embankment section to facilitate hole layout and faster movement of equipment. Rock was drilled using water-powered down the hole (WDTH) rotary percussion drills, and boreholes were monitored by high resolution imaging equipment. Grout used in this phase was a balanced, stable mixture of water, cement, hydrated bentonite slurry polymer, and superplasticizer, which produced very little bleed and was highly resistant to pressure filtration. The objective of grouting was to provide a curtain with maximum permeability of 10 Lugeons in area of the proposed cutoff wall, and 3 Lugeons in the rock below the cutoff wall. The use of advanced drilling equipment, grout, and computer monitoring significantly accelerated the grouting program, thus accomplishing the Phase 3 goal of interim risk reduction.

Phase 4 – 2009 to 2011: Embankment Contact Grouting and Completion of Deep Grout Curtain. Phase 4 involved five separate tasks:

1. An LMG double-line pre-grouting program for the embankment/foundation rock interface zone to reduce possibility of a major slurry loss

2. Main embankment – curtain grouting program using double-line HMG
3. Right rim – curtain grouting program using single-line HMG
4. Over-water work, near embankment dam to concrete monolith contact area. This program was modified and became a fan grouting program.
5. Critical Area 1 Curtain Grouting Program using HMG

Phase 4 LMG and HMG grouting together included drilling a total of about 274,000 linear feet embankment and rock and injecting 375,000 gallons of grout. Barrier wall construction was successful in the critical area without slurry losses or other problems. The 1700 feet long gallery and plaza were also grouted as part of a separate Phase 5.

The successes of various grouting programs undertaken for the Wolf Creek Dam are attributed to the evolution of different technological aspects – drilling method, grout mix, monitoring technology, control and analysis, grout injection pressures, and closure criteria. The latter phases used the most advanced techniques such as WDTH rotary percussion drill, balanced, stable HMGs, colloidal grout mixing, computerized analysis and control with CAD, and zone-specific injection pressures as opposed to rule-of-thumb.

3.0 CONSTRUCTION AND MATERIALS

This section addresses general grouting equipment and materials used on grouting projects.

3.1 Construction Equipment

This section provides a discussion of equipment used for grouting, and rock and soil drilling. There are basically three types of rock drilling: 1) high rotation speed/low torque rotary drilling, 2) low rotation speed/high torque rotary drilling, and 3) rotary percussive drilling.

3.1.1 High Rotation Speed/Low Torque Rotary Drilling

High rotation speed – low torque drilling is typically used for grout holes up to 3 inches diameter to depths of 160 – 800 ft. Relatively light drill rigs can be used to extract core samples when using a core barrel system, or can also be used simply to drill grout holes, using “blind” or “plug” diamond impregnated bits, as illustrated in Figure 8-34.

http://www.jksboyles.co.uk/drillbits.html (top) and http://www.drillingcourse.com/2016/01/naturel-diamonds-drilling-bits.html (bottom)

Figure 8-34. Diamond drilling tools.

Due to slow rates of penetration and deviation issues, such methods are rarely used nowadays for grout hole production drilling.

Advantages of high speed rotary drilling include the following:

- The same equipment can be used for both investigatory and grout hole drilling.

- Continuous or intermittent exploration of the rock is possible over the entire length of the hole.
- Drilling can be performed to relatively great depths, but not typically required
- No or limited clogging of the rock fissures typically occurs. Cuttings are removed from the hole with the flush water.
- Vibration is minimized; hence, this is the preferred technique for drilling through existing masonry and brickwork.
- It is possible to drill in all kinds of rock.
- It is possible to use most power alternatives to drive the equipment (i.e., air, electricity and diesel).
- Rotary drill bits produce smooth hole walls that make subsequent packer installation easier.
- Good penetration speeds can be achieved in soft formations.

3.1.2 Low Rotation Speed/High Torque Rotary

Low rotation speed – high torque drilling is used with heavier and more powerful rigs to drill holes of greater diameter to considerable depths. The penetration rate also depends on the amount of thrust applied to the bit. A variety of carbide drilling tool bits are shown in Figure 8-35.

http://cnforsuntools.en.made-in-china.com/product/XSsnFCYOiHVv/China-Tc-Carberit-Core-Drill-Bit-for-Soft-Rock-Formation.html (left), http://cnforsuntools.en.made-in-china.com/productimage/JvSnUjouhcWp-2f1j00eZAEhLIJkmoQ/China-Three-Wing-Tungsten-Carbide-Drag-Bit.html (center), and http://m.made-in-china.com/product/Carbide-Tooth-Three-Roller-Bit-TCI-Tricone-Bit-26-704877402.html (right)

Figure 8-35. Carbide drilling tools.

3.1.3 Rotary Percussive

Rotary percussive drilling uses drill bits that are both percussed and rotated (cross or button). In general, the percussive energy determines the penetration rate either with a top hammer, where the drill rods are rotated and percussed by the drill head on the rig, or with a down-the-hole hammer, where the (larger diameter) drill rods are only rotated by the drill head and compressed air or high pressure water is fed down the rods to activate the percussive hammer mounted directly above the bit.

Top hammer drilling is performed at rotation speeds of approximately 60 to 120 rpm in hole diameters seldom above 4 inches. Grout hole depth is limited to approximately 200 feet by power, and by hole deviation concerns.

Down-the-hole drilling is performed at approximately 10 to 60 rpm in hole diameters of 3.3 inches and above, to depths of over 330 feet.

Percussion-drilled grouting holes should be flushed by water to avoid the cuttings clogging the fissures. Especially below the water table, air flushing is risky as a sludge may be formed that closes off the fissures that will have to be grouted at a later stage. Thus, air-powered, down-the-hole drilling is not acceptable for fissure grouting applications, although the speed and straightness benefits of the principle can still be exploited by the new generation of water powered hammers (Bruce et al. 2013).

Advantages of percussion drilled grout holes include the following:

- Higher and more consistent penetration rates can be maintained in rock, as compared to other methods.
- Smaller and lighter drill rigs can be used; these are easily moved from hole to hole on the surface.
- Low drilling costs can be achieved, as compared with rotary drilling.
- It is possible to optimize the equipment for drilling through layers of different hardness and thickness.

Top hammer drilling is the most common and generally also the least expensive method, but it limits the hole depth and is subject to the greatest hole deviations. This means an increased number of holes and increased costs, as well as lower quality. Down-the-hole hammer drilling results in straighter and deeper holes with relatively constant penetration rates. Hole linearity and drill access restraints may also have significant impact on choice. In principle, the prime controls over the choice of drilling method should ideally be related to the geology, hole depth, and diameter.

In the United States, rock drilling is largely and traditionally conducted by rotary methods, although the insistence on diamond drilling is no longer so prevalent. However, top drive rotary percussion is growing in acceptance due to the increasing availability of higher powered diesel and hydraulic drill rigs using water or foam flush. Air-flush methods are applicable for drilling grout holes to locate and fill large voids, such as karstic features, and water powered, down-the-hole hammers offer significant cost and technical advantages for rock fissure drilling.

3.1.4 Rock Drilling Summary

The drilling method selected must:

- drill a straight hole,
- protect the hole walls from caving in,
- produce drill cuttings of such a size that they can be flushed out without closing the fissures in the ground or blocking the subsequent grouting, and
- be cost-effective.

Grout holes should be drilled such that they intercept as many fissures in the rock mass as possible. Where this requirement is difficult to achieve, the spacing must be reduced instead to ensure that fissure planes with an unfavorable orientation to the grout holes will be grouted as efficiently as possible. Hence, we always have at least two rows of holes, inclined towards the left and right, respectively.

Larger diameter cores provide more reliable information about the ground. Because of the stiffness of the drill string, larger hole diameters in general result in straighter but more expensive holes. The setting of packers is more expensive and also more difficult in larger diameter holes, and final backfilling costs higher. Hole straightness is important to address, since excessive deviation may leave unpenetrated “windows” in the curtain, leading to incomplete treatment. For greater hole depths, guide rods (centralizers) and drill string supports may be used, together with thicker walled drill rods.

Some commonly attainable hole deviation limits are as follows:

- High speed rotary drilling: normally 2 to 5% to depths of 260 feet
- Top hammer drilling: long holes – 15 to 20% (with guide rods, under 5% can be reached); shallow holes, down to 40 to 50 feet – under 5% is possible also without guide rods. Long top hammer holes drilled with guide rods incur a high risk of getting stuck.

- Down-the-hole drilling: typically less than 2%, and less than 1% with high standards of workmanship.

The size of the drill cuttings can vary from muddy clay to flaky gravel. Different drilling methods produce cuttings that vary in form, size, and shape. All holes drilled for grouting must be cleaned carefully of drill cuttings and loose ground material lodged in the cracks. In general, this is done by high pressure water flushing from the bottom up toward the collar of the hole. For guidance on flush, refer to Weaver and Bruce (2007).

3.1.5 Measurement While Drilling (MWD)

MWD is a method for continuous recording of various drilling parameters. It measures the drill rig's behavior during the drilling operation, and is used to provide a broad categorization of the ground. The measured variables can be depth, rate of penetration (ROP), weight on bit (WOB), feed force, rpm, torque, flush water flow, flush water pressure, and time. MWD is usable both on percussive and rotary drill rigs.

Depending on the ground, there will be a variation in the drilling parameters that are recorded, mainly the variation of the hydraulic flow, pressure parameters, and penetration rate due to geological variations. Various geological conditions can, therefore, produce similar hydraulic characteristics. The measured parameters should be correlated with the drilled core sample from a drill hole nearby.

The variables, feed force and rpm are set by the driller. The variables ROP, torque, and rate of penetration are dependent on the formation being drilled. The variables flush water flow and flush water pressure are dependent on the driller, the drill equipment, and the formation being drilled.

For example, the dividend of the flush water flow and the flush water pressure can be used to locate major fissures and cracks: when the drill bit hits a fissure, the pressure will drop and, at the same time, the flow will increase. This is due to the inflow of the flush water into the fissure. The data may be electronically generated or similar data may be recorded manually, and is always of great value in helping to understand the ground and the changes being effected on it by each successive phase of drilling and grouting. While the use of automated MWD is becoming increasingly common, it does not completely replace normal, manual logging. Both sets of data should be studied when attempting to analyze the ground.

3.1.6 Soil Drilling Methods

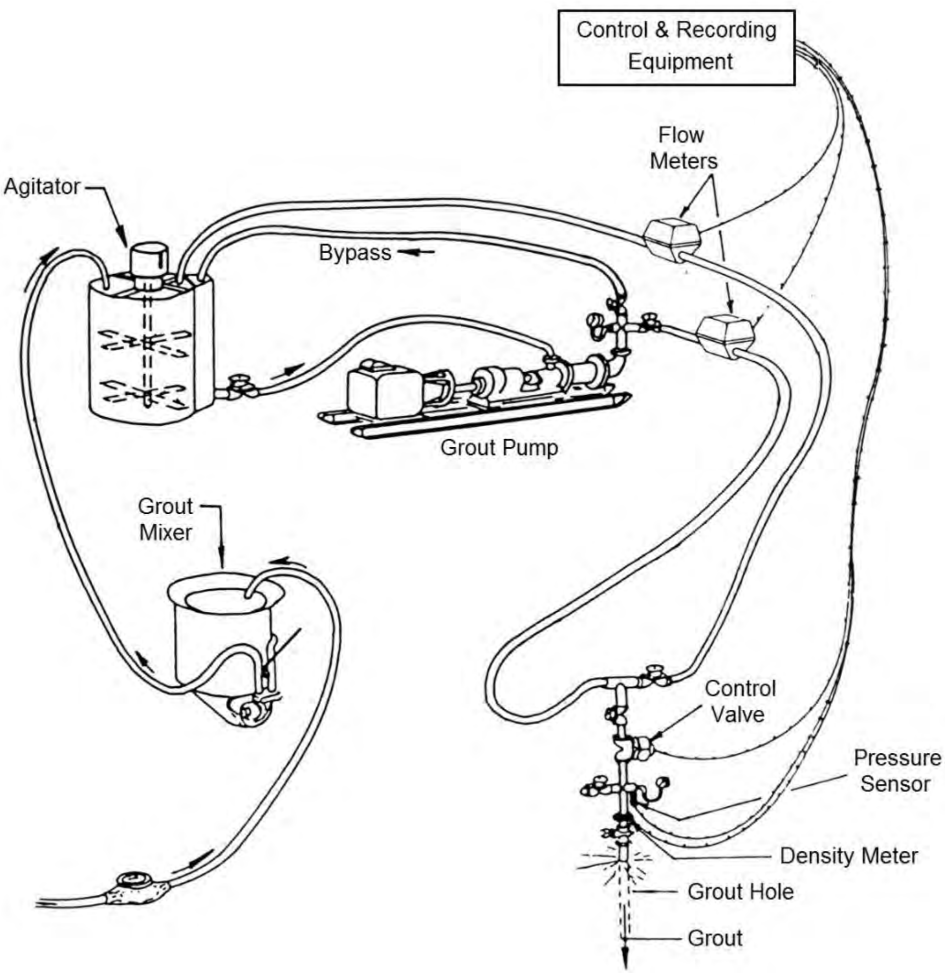
There is a wide range of overburden drilling systems to drill soils for soil grouting. The choice of method should satisfy the geometric requirements of the drilling, and be consistent with the geotechnical and environmental challenges of the soil.

The logic of choice is perhaps even more obscure than in rock drilling, and history and habit have ensured that not all methods are used by any one contractor, or in any one geographical region. Hollow stem augers are common around the Great Lakes and on the West Coast, while simple flushed casings and rotary duplex are favored in the East. The emergence of foreign-backed drill rental companies offering percussive duplex and double-head duplex capabilities has spread these techniques nationwide. Percussive duplex (eccentric) is in general decline for routine production grout holes, although it is still regarded in certain quarters as the premier soil drilling method in very difficult conditions. This has recently been replaced by the “Rotoloc” system of CRI, which uses “wings” to oversize the hole and thereby permitting the casing to be introduced with minimal torque.

The choice of flush is critical, especially for cohesionless materials below the water table: air should never be used in such circumstances. Most recently, sonic drilling has become very popular, especially for applications demanding absolutely minimal damage to the surrounding soil (e.g., penetrating through an existing embankment dam). This method is fast, reliable, and uses no, or very little, flush. Further details are provided in Bruce (2003).

3.1.7 Grouting Equipment

Many types of grouting equipment are commercially available and are used routinely for grouting operations of different types and scale. Each major rock and soil grouting technique basically demands its own specialized equipment. However, main components are grout-mixing equipment of a capacity adequate for the job and that mixes grout to a uniform consistency; a storage tank capable of continuous agitation of the grout to prevent settlement and segregation; a pump capable of precise pressure and volume control; appropriate grout parameter recording equipment; and a system of grout lines with a header for injecting grout into the hole as desired. Suitable packers, gauges, valves, and accessories are also required. A schematic layout for an HMG (high mobility grout) injection application is shown in Figure 8-36. Today, much of this is replaced with a sophisticated plant/pump operation that is computer controlled.



Houlsby 1990

Figure 8-36. Schematic layout for HMG injection.

The grout mixer and agitator need not be of the same volume capacity. Where high grout takes are anticipated, two mixers may be arranged to discharge into the same storage tank. Both the mixer and the agitator should continuously agitate the grout until it is either injected or wasted. For HMGs, high-speed, high-shear (colloidal) grout mixers are far superior to standard slow-speed mechanical mixers because they produce grouts of greater uniformity and quality more quickly. Bentonite is mixed in a separate mixer and must be fully hydrated before being introduced into the grout mixer. Water is metered into the mixers, and the meter should be calibrated in liters and be large enough for easy reading. The use of the metric system is numerically advantageous in grouting calculations for batching.

Various types of pumps are used, again depending on the application. The pump should be specified based on the individual job requirements. Either piston pumps or progressive cavity pumps are used for HMG, concrete pumps for LMG grouting (modified as necessary), and custom built equipment for chemical and jet grouting.

Technique-specific aspects regarding equipment are addressed in Section 2. Typical examples of drilling and grouting equipment are shown in Figure 8-37 through Figure 8-47.

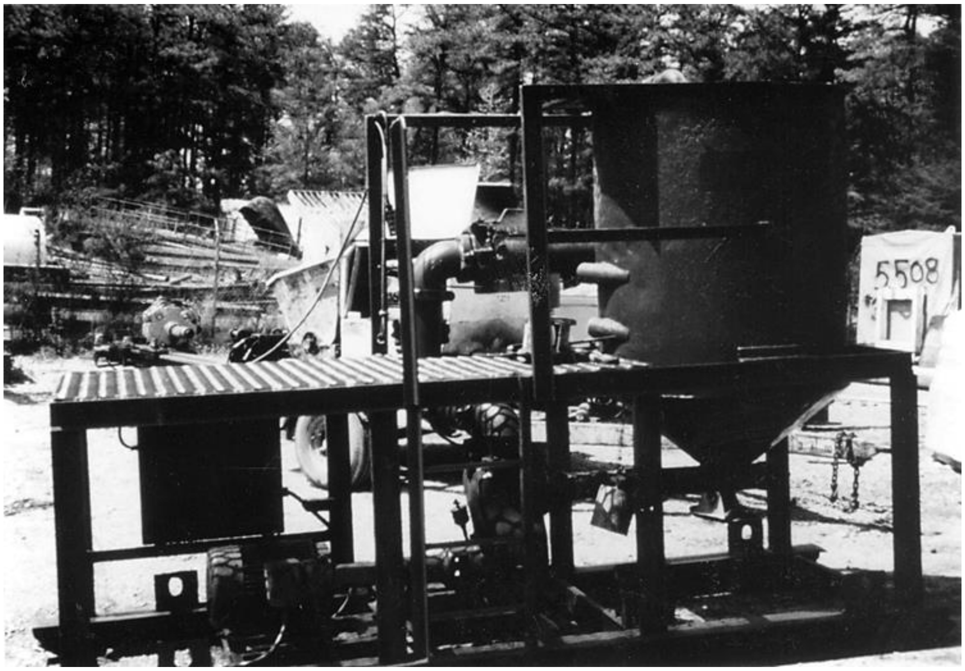


Figure 8-37. Electric-powered, high-shear HMG mixer.

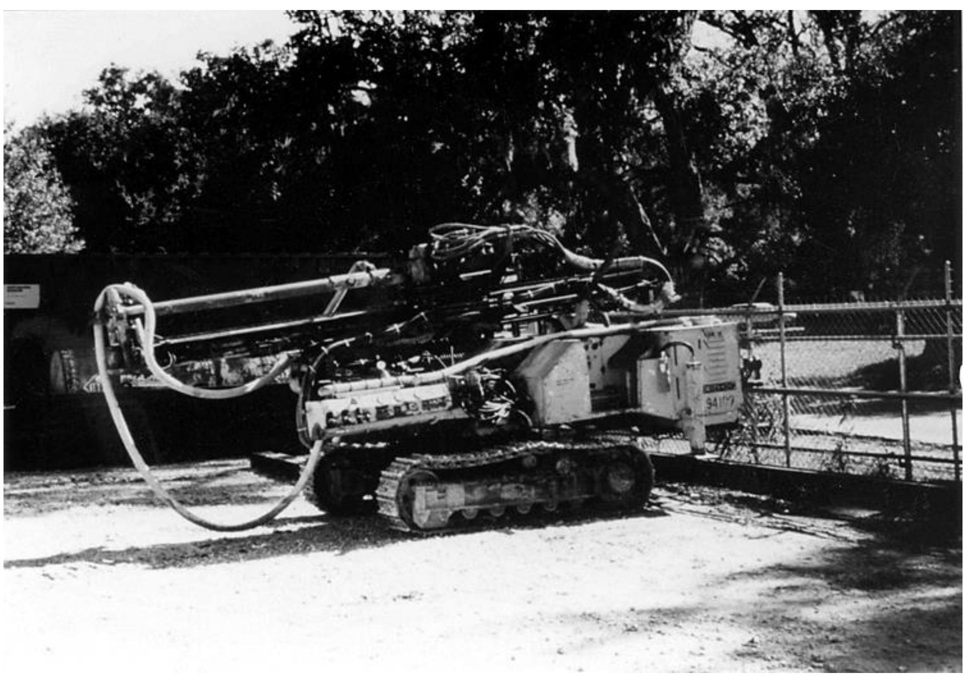


Figure 8-38. Rotary/rotary-percussion diesel hydraulic track drill.

Figure 8-39. Small compaction grout batcher.

Figure 8-40. Larger on-site grout batching plant for compaction grouting.

Figure 8-41. Compaction grout pump.

Figure 8-42. Compartmentalized tanker for raw chemical grout components.

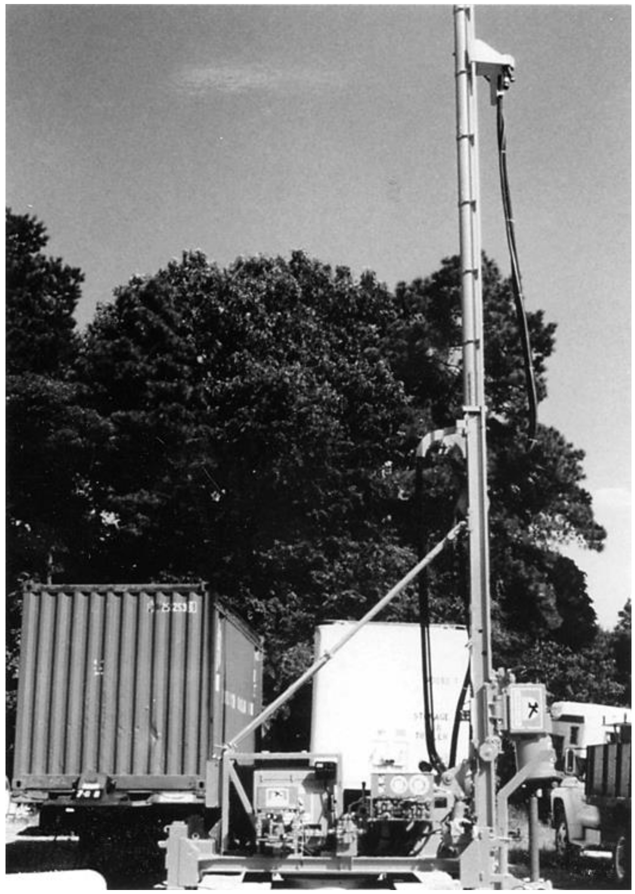


Figure 8-43. Jet grouting rig.

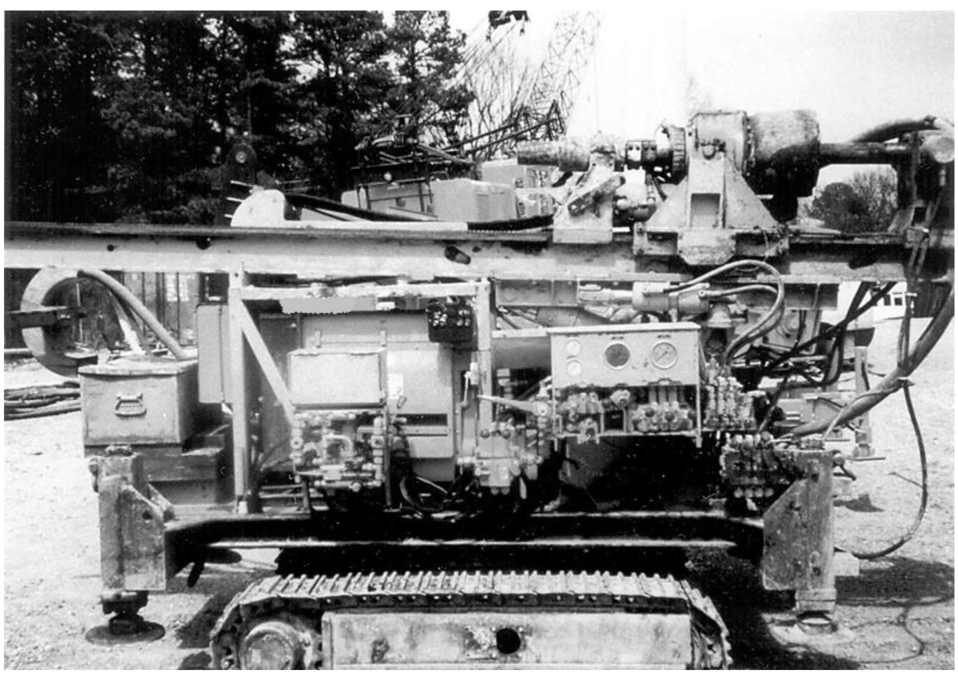


Figure 8-44. Micropile rig – drill mast at rest position.

Figure 8-45. Pumping station for HMG grouting.

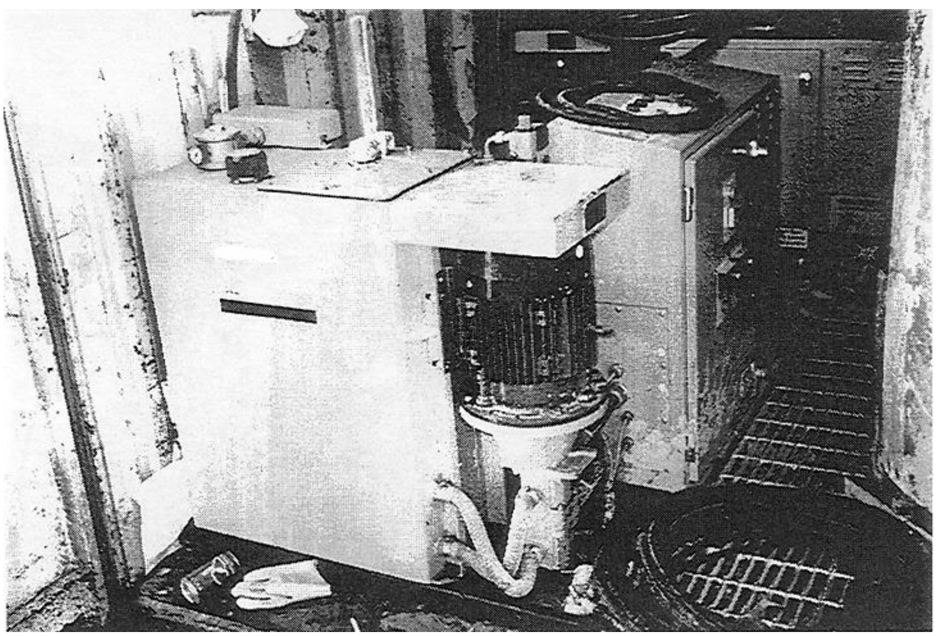


Figure 8-46. High pressure HMG pumping unit.

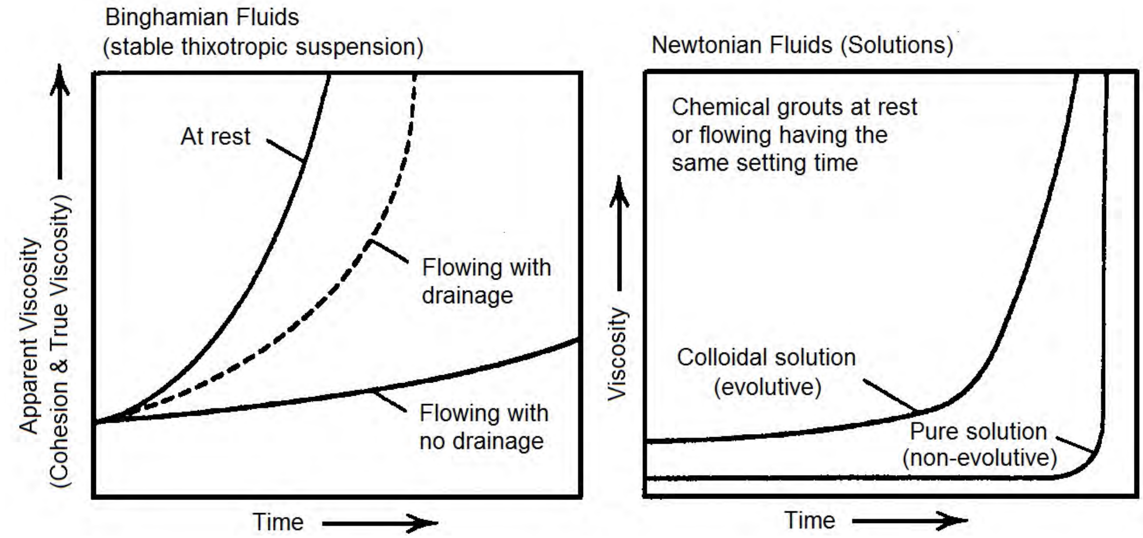
Figure 8-47. Hydraulic crawler rig for lime injection.

3.2 Materials

Grouting materials can be classified into the following four categories, listed in order of increasing rheological performance and cost (Bruce et al. 1997):

1. Particulate (suspension or cementitious) grouts, having a Binghamian performance (See Figure 8-48 left).
2. Colloidal solutions, which are evolutive Newtonian fluids in which viscosity increases with time (See Figure 8-48 right).
3. Pure solutions, being non-evolutive Newtonian solutions in which viscosity is essentially constant until setting, within an adjustable period.
4. Miscellaneous materials.

Category 1 comprises mixtures of water and one or several particulate solids such as, cement, fly ash, clays, or sand. Such mixes, depending on their composition, may prove to be stable (i.e., having minimal bleeding) or unstable when left at rest. Stable, thixotropic grouts have both cohesion and plastic viscosity increasing with time at a rate that may be considerably accelerated when excess pressure is applied.



Mongilardi and Tornaghi 1986

Figure 8-48. Rheological characteristics of major families of grouts: category 1 (left), categories 2 and 3 (right).

Category 2 and 3 grouts are now commonly referred to as solution or chemical grouts, and are typically subdivided on the basis of their component chemistries such as silicate based (Category 2) or resins (Category 3). The outstanding rheological properties of certain Category 3 grouts, together with their low viscosities permit permeation of soils as fine as silty sands ($k = 4 \times 10^{-5}$ inch/second).

Category 4 comprises a wide range of relatively exotic grout materials, which have been used relatively infrequently, and only in certain industries and markets. Nevertheless, their importance is growing due to the high performance standards that can be achieved when they are correctly used. The current renaissance in the use of hot bitumen grouts for fast flow sealing is a good example (Bruce 2003). Table 8-10 provides a summary of characteristics of Category 2 and 3 grouts used for water control, and their relative costs.

Table 8-10. Characteristics of Grout Material for Water Control Purpose

Description	Viscosity (cP) – w/c ratio	Toxicity	Strength	Material Cost/Quart	Remarks
Colloidal Solution: Silicates	Low (> 6 cP)	Low	Med	> $0.13	Penetrates fine fissures
Solution Grout: Lignosulfites	Med (> 8 cP)	High	Low	> $0.26	Penetrates fine fissures
Solution Grout: Polyurethane	High (> 400 cP)	High	High	> $1.32	Penetrates large fissures
Solution Grout: Acrylamides	Low (1.2 cP)	High	Low	> $0.53	Penetrates very fine fissures
Solution Grout: Acrylates	Low (1.2 cP)	Low	Low	> $0.53	Penetrates very fine fissures

3.2.1 Particulate Grouts

Due to their basic characteristics, and relative economy, these grouts remain the most commonly used for both routine waterproofing and ground strengthening. The water-to-solids ratio is a prime determinant of their properties and basic characteristics such as stability, fluidity, rheology, strength, and durability. The following broad subcategories can be identified:

- Neat cement grouts
- Clay/bentonite-cement grouts
- Grouts with fillers
- Grouts for special applications
- Grouts with enhanced penetrability

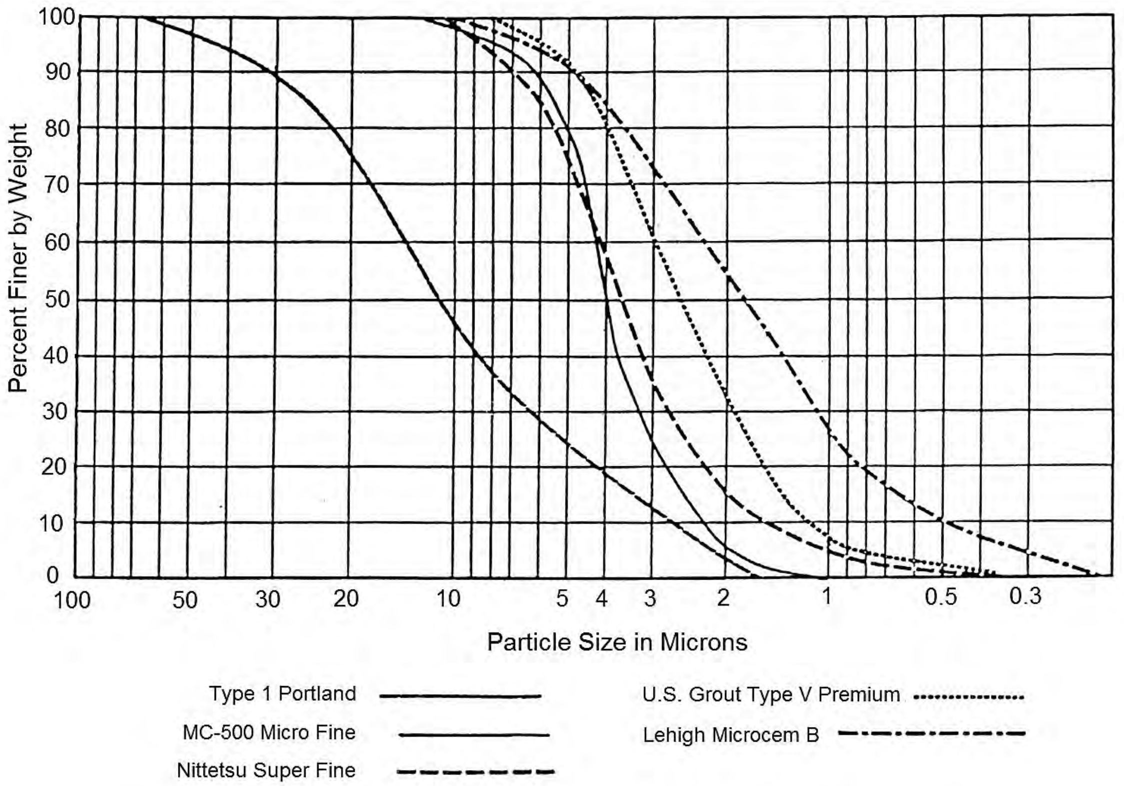
Typically in the United States, water/cement (w/c) ratios have been expressed as a volumetric ratio rather than a weight ratio. Given the increased use of semi-automatic batching equipment, it is easier to work in weight ratios. For example, a grout with w/c = 1 by weight comprises approximately 12 gallons (100 Liters) and 242.5 lbs. (100 kg) of cement. Additives and admixtures are normally expressed also as a weight ratio to cement. As a rule

of thumb, to obtain water/cement ratios by volume, multiply the water/cement ratio (by weight) by 1.5.

Portland cements are the most common and best-known cements used worldwide as the basic ingredient for particulate grouts. The following provides a general description:

- Type I Portland cement is accepted as the general purpose cement for use in the majority of grouting applications, when the special properties of other types are not required.
- Type II Portland cement is manufactured to resist moderate sulfate attack and to generate a slower rate of heat of hydration than that exhibited by Type I.
- Type III Portland cement is used when higher early strengths are desired. It is considered for phases of grouting applications to be put into service quickly or for emergency repairs. Since particle size is smaller than in other types, it is sometimes specified for grouting slightly finer fissures.
- Type IV Portland cement generates less heat during hydration than Type II, and develops strength at a much slower rate than Type I. It is considered for use in large, mass grout placements, when high temperatures of heat of hydration are not acceptable.
- Type V Portland cement is manufactured for use in grout exposed to severe sulfate action. It is used principally when a high sulfate content is present in soils or groundwater.

Microfine cements are simply finer ground versions of both Portland and blast furnace slag cements. Typically, the maximum particle size is less than 3.2×10^{-4} inches (8 microns), with the bulk being less than 1.6×10^{-4} inches (4 microns). Examples of the gradation curves from some of the many types now available in the U.S. are shown in Figure 8-49.



Units: 1 micron = 4 × 10^{-5} inches

Warner 1999

Figure 8-49. Grain size distribution for cement types.

Note that many particulate grouts are unsuited for sealing high flow, high head conditions. They will be diluted or washed away prior to setting in the desired location. Low mobility grouts can be classified in the third subgroup, and can be used for seepage reduction under appropriate conditions.

When the grout is forced to enter a small aperture under pressure, water can be expelled from the grout depending on its composition, resulting in development of a cementitious filter cake at the borehole wall. The filter cake eventually blocks off the aperture such that grout can no longer enter the aperture. This tendency of grout to lose water during injection into an aperture under pressure is quantified by the "Pressure Filtration Coefficient," K_{pf}, and by the filter cake growth coefficient, K_{pc}, which are calculated as follows (Weaver and Bruce 2007):

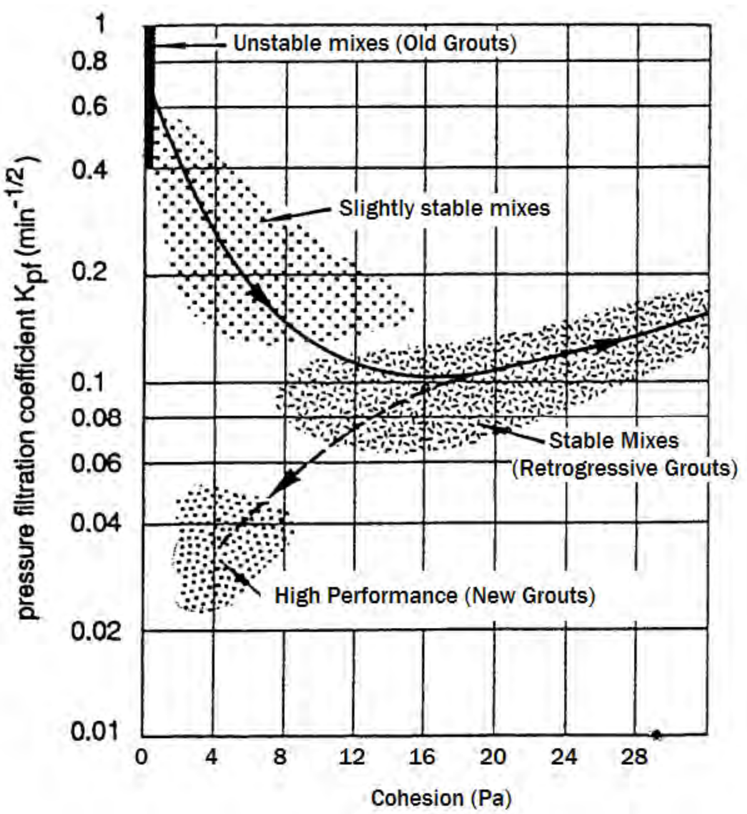
$$K_{pf} = \frac{V_f}{V_i\sqrt{t}}\ \text{min}^{-1/2} \qquad \text{and} \qquad K_{pc} = \frac{h}{\sqrt{t}}\ \text{inch} \times \text{min}^{-1/2} \qquad \text{[Eq. 8-7]}$$

where,

V_f = Volume of filtrate, fl. oz. (or ml)

V_i	=	Initial volume of sample, fl. oz. (or ml)
h	=	Thickness of filter cake, inch
t	=	Test time, minutes

A low pressure filtration coefficient that minimizes the increase in apparent viscosity is required to increase the penetrability of the grout. Figure 8-50 shows the variation in pressure filtration coefficient as a function of cohesion.



Units: 1 Pa = 1.45×10^{-4} psi

Weaver and Bruce 2007

Figure 8-50. Pressure filtration coefficient versus soil cohesion.

3.2.2 Colloidal Solutions

Colloidal solutions comprise mixtures of sodium silicate and reagent solutions, which change in viscosity over time to produce a gel. Sodium silicate is an alkaline, colloidal aqueous solution. It is characterized by the molecular ratio, R_p, and its specific density, expressed in degrees Baumé (ºBé). Typically R_p is in the range 3-4, while specific density varies from 30-42º Bé. Reagents may be organic or inorganic (mineral). The former cause a saponification hydraulic reaction that frees acids and can produce either soft or hard gels,

depending on silicate and reagent concentrations. Common types include monoesters, diesters, triesters, and aldehydes, while organic acids (e.g., citric) and esters are now much less common. Inorganic reagents contain cations capable of neutralizing silicate alkalinity. In order to obtain a satisfactory hardening time, the silicate must be strongly diluted, and so these gels are typically weak and, therefore, of use only for waterproofing. Typical inorganic reagents are sodium bicarbonate and sodium aluminate.

The relative proportions of silicate and reagent will determine by their own chemistry and concentration the desired short- and long-term properties such as gel setting time, viscosity, strength, syneresis, and durability, as well as cost and environmental acceptability.

In general, sodium silicate grouts are unsuitable for providing permanent seepage barriers against high-flow/high-head conditions because of their relatively long setting time (20 – 60 minutes), low strength (less than 290 psi), and poor durability. However, they may prove locally acceptable for temporary applications, say less than a few months. Sodium silicate solution without reagent may be used to accelerate the stiffening of cementitious grouts, a traditional defense against fast flows in small orifices.

3.2.3 Pure Solutions

Resins are solutions of organic products in water, or a non-aqueous solvent, capable of causing the formation of a gel with specific mechanical properties under normal temperature conditions and in a closed environment. They exist in the following forms, characterized by their mode of reaction or hardening:

- Polymerization: activated by the addition of a catalyzing element (e.g., poly-acrylamide resins).
- Polymerization and Polycondensation: arising from the combination of two components (e.g., epoxies, aminoplasts).

In general, setting time is controlled by varying the proportions of reagents or components. Resins are used when particulate grouts or colloidal solutions prove inadequate, for example when the following grout properties are needed:

- particularly low viscosity
- very fast gain of strength (a few hours)
- variable setting time (few seconds to several hours)
- superior chemical resistance
- special rheological properties (pseudoplastic)

- resistance to high groundwater flows

Resins are used for both strengthening and waterproofing where durability is essential, and the above characteristics must be provided. Four categories can be recognized: acrylic, phenolic, aminoplastic, and polyurethane, as indicated in Table 8-11. Chrome lignosulfonates are not discussed, because of the environmental damage caused by the highly toxic and dermatitic components.

Table 8-11. Uses and Applications of Resins

Type of Resin	Nature of Ground	Use/Application
Acrylic	Granular, very fine soils Finely fissured rock	• Waterproofing by mass treatment • Gas tightening (mines, storage) • Strengthening up to 220 psi Strengthening of a granular medium subjected to vibrations
Phenol	Granular, very fine soils	• Strengthening
Aminoplastic	Schists and coals	• Strengthening (by adherence to materials of organic origin)
Polyurethane	Large voids	• Formation of a foam that forms a barrier against running water (using water-reactive resins) • Stabilization or localized filling (using two-component resins)

Of these four subclasses, only the two following groups of polyurethanes are usually appropriate for grouting:

- Water-Reactive Polyurethanes: Liquid resin, often in solution with a solvent or in a plasticizing agent, possibly with added accelerator, reacts with groundwater to provide either a flexible (elastomeric) or rigid foam. Viscosities range from 0.034–0.067 pound/foot/second. They may be either
    - Hydrophobic: react with water, but repel it after the final (cured) product has been formed, or
    - Hydrophilic: react with water, but continue to physically absorb it after the chemical reaction has been completed.
- Two Component Polyurethanes: Two compounds in liquid form react to provide either a rigid foam or an elastic when supplemented with a polyisocyanate and a

polyol. Such resins have viscosities from 0.067–0.67 pound/foot/second and strengths as high as 300 psi. A thorough description of these grouts was provided by Naudts (1995).

3.2.4 Miscellaneous Grouts

These grouts are essentially composed of organic compounds or resins. In addition to waterproofing and strengthening, they also provide very specific qualities, such as resistance to erosion or corrosion, and flexibility. Their use may be limited by specific concerns, such as toxicity, injection and handling difficulties, and cost. Categories include hot melts, latex, polyesters, epoxies, furanic resins, silicones, and silacsols. Some of these (e.g., polyesters and epoxies) have little or no application for ground treatment. Others, such as latex and furanic resins, are even more obscure and are very infrequently encountered in practice.

For certain cases in seepage cut off, hot melts can be a particularly viable option. Bitumens are composed of hydrocarbons of very high molecular weights, usually obtained from the residues of petroleum distillation. Bitumen may be viscous to hard at room temperature, and have relatively low viscosity (0.01 to 0.034 pound/foot/second) when hot (typically in excess of 400°F). It is used in particularly challenging water-stopping applications, remains stable with time, and has good chemical resistance. Contemporary optimization principles require simultaneous penetration of the placed bitumen mass by stable particulate grouts to ensure good long-term performance of the system (Bruce 2003).

Silacsols are also of considerable potential, which are solution grouts formed by reaction between an activated silica liquor and a calcium-based inorganic reagent. Unlike the sodium silicates discussed above, aqueous solutions of colloidal silica particles disperse in soda, and the silica liquor is a true solution of activated silica. The reaction products are calcium hydrosilicates with a crystalline structure similar to that obtained by the hydration and setting of Portland cement, i.e., a complex of permanently stable crystals. This reaction is not, therefore, an evolutive gelation involving the formation of macromolecular aggregates, but is a direct reaction on the molecular scale. This concept has been employed in Europe since the mid-1980s with consistent success in fine-medium sands (Bruce 1988). The grout is stable, permanent, and environmentally compatible. Other important features, relative to silica gels of similar rheological properties are:

- far lower permeability
- far superior creep behavior of treated sands for grouts of similar strength (290 psi)
- permanent durable filling is assured, even if an unusually large pore space is encountered, or a large hydrofracture fissure is created

4.0 PROJECT PLANNING

Proper planning is essential for the completion of a successful grouting program. The planning process includes several phases from determining project requirements, preliminary investigations, plans and specifications, design to post-construction monitoring, performance assessment and quality assurance and verification of the final product. This section provides grouting project users a suggested sequence of steps to plan and implement a successful grouting application.

4.1 Project Planning Steps

4.1.1 Step 1: Determine Project Performance Requirements

In order to determine the requirements of the grouting activity, a preliminary analysis of the desired qualitative and quantitative levels of performance should be conducted. This should include the extent of water control or water proofing to be achieved by grouting should be determined by measuring water flow and water pressure. The required structural improvement of problematic geomaterial – soils and/or rocks, should be well-defined in terms of compressive or shear strength or increased stiffness of the grout and the grouted mass.

4.1.2 Step 2: Assess the Adequacy of the Subsurface Information

Based on the information assembled in step 1, determine the adequacy of the existing subsurface information to assess the extent of the problematic condition. This would include treatment area and depth as well as variability of subsurface strata and index and performance parameters to evaluate existing conditions and the predicted performance before and following ground improvement treatment.

4.1.3 Step 3: Identify and Assess General Site Conditions

General site conditions such as construction and operational space, existing structures (both overhead and sub-surface), constructability and environmental constraints should be investigated prior to design of the grouting program.

4.1.4 Step 4: Technical Feasibility

All possible solutions to the problem, i.e. selection of the applicable grouting types (permeation grouting, jet grouting or others) should be identified based on technical feasibility. The technical feasibility assessment is a critical planning step and should be focused on the specific expectations and outcomes to the extent possible. The feasibility of a grouting solution is directly related to the existing geomaterial parameters and the degree of

improvement necessary to meet the project performance requirements. This is a good time to first consult with specialists (consultants and/or contractors).

Another critical aspect of this step is to evaluate qualitative and quantitative methods which will be used to test the end product following grouting, to accept the improved subsurface conditions. As discussed in each of the previous sections of this Technical Summary of grouting, it is generally difficult to define success quantitatively. Even when in situ or other field tests are used during and following grouting, these often require judgment and interpretation while at the same time, specifications and contracting documents must be clear and specific.

4.1.5 Step 5: Preliminary Design and Cost Estimates

The project owner must conduct a preliminary design in order to develop a preliminary cost estimate. The GeoTech Tools website and sections of this grouting technical summary provide general guidance on both design and costs. Even when comprehensive grouting design information is available, the cost estimate should include a significant contingency percentage to address owner directed changes in scope and grout volumes based on field monitoring of the contractor's means and methods. Risks associated with a grouting program and some of their probable causes are listed in Table 8-12 below.

Table 8-12. Risks Associated with Grouting

Type of Risk	Causes
Schedule and Budget Risk	• Contractual issues between general and specialty contractor • Late notice to proceed • Weather delays • Site access problems • Material supply and storage problems • Equipment availability issues • Different site conditions
Geotechnical Issues and/or Poor Soil Conditions	• Slope instability • Settlement • Liquefaction • Contamination • Problem soils, e.g., expansive clays • Impact on adjacent, existing structures

4.1.6 Step 6: Specifications, Contract Documents and Quality Assurance Requirements

The design of a grouting project requires the development of contracting documents which are often written as a performance based specification format which typically includes some means and methods controls, but largely relies on the qualifications and experience of the grouting specialty contractor. These specifications should include a prequalification requirements or a documented experience record as a requirement as a submittal following contract award.

Quality Assurance requirements should address the grouting contractor's quality control procedures for the principal grouting aspects (grout material, grout pipe drilled and placement of the grout material). The contractor is also often responsible to conduct sampling and testing procedures which assure the end product has met the project performance requirements, but an outside agency should be engaged. The owner may independently conduct some level of independent testing and monitoring to validate that the contractor has followed the proposed QA process and details.

4.1.7 Step 7: Construction Monitoring and End Results

The owner should be actively involved in monitoring the progress and schedule during construction, and evaluating the results of the grouting program. One reason for this proactive approach is to determine in real time whether the interim results and testing procedures meet the owner's expectation and if they don't, to implement changes to acceptance procedures or the contractor's means and methods. An outside consultant is often employed to perform this task.

The above steps can be summarized as designer and contractor responsibilities as shown below in the following sequential steps:

- Establishing specific objectives for the grouting program (designer)
- Defining the geometric and geotechnical project conditions (designer) and the properties of the treated soil/rock
- Developing an appropriate grouting program design and companion specifications and contract documents (designer)
- Planning the grouting equipment needs and procedural approach (contractor)
- Monitoring and evaluation of the grouting program (designer, contractor)

The planning process is shown in the flowchart in Figure 8-51.

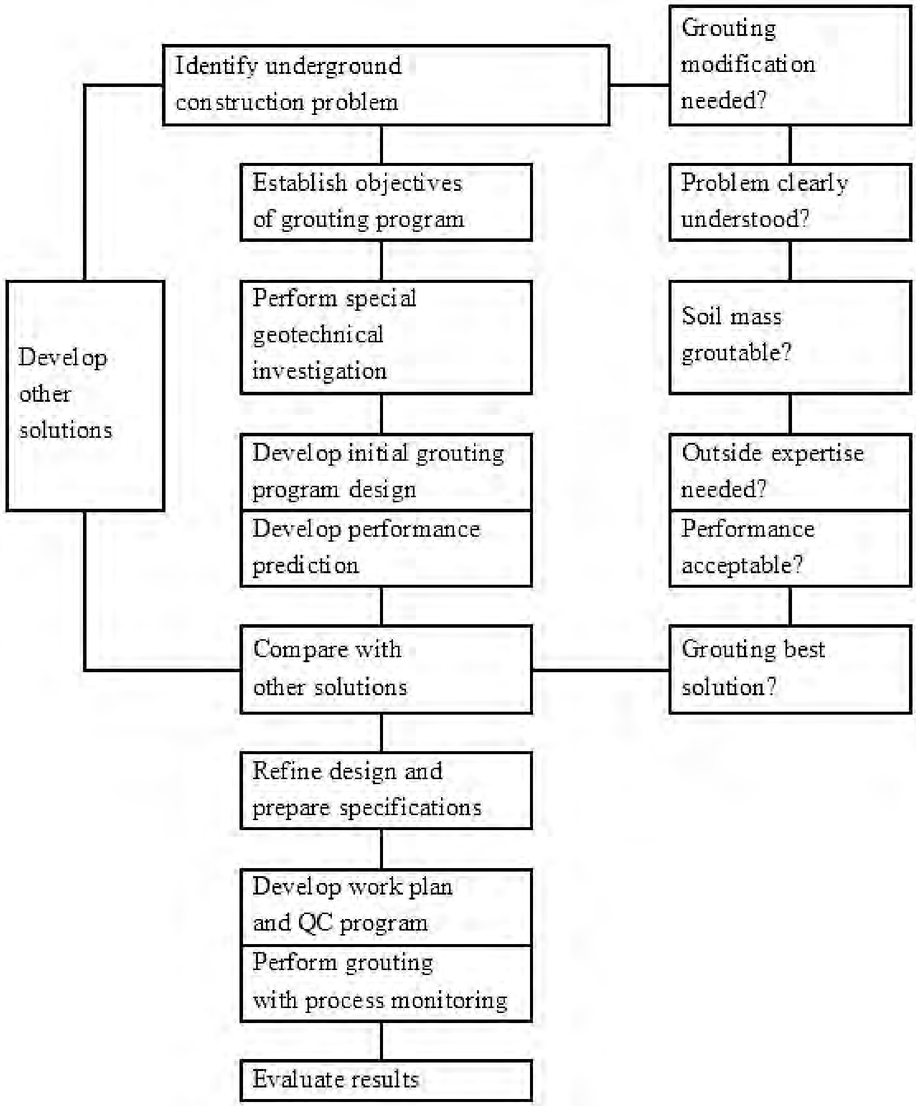


Figure 8-51. Grouting decision flowchart.

4.2 Pregrouting Subsurface Investigations

Pregrouting subsurface investigation programs will normally require more than the usual number of borings, and should include continuous samples and laboratory tests. These tests should include grain size analysis, density, permeability, pH, and other soil index properties. The purpose of the subsurface investigation is to define the limits and characteristics of the geotechnical situation to be solved by the grouting process.

Equally important is the clear identification of the geological subsurface conditions that will control and permit the success of the grouting approach. This includes a thorough knowledge of the stratigraphy, environment, and groundwater regime.

Stratigraphy is the variation in soil properties, especially permeability, strength, and density of the grouting zone, which is an important controlling factor in the design and effectiveness of the grouting process. A higher sampling frequency should be used to identify separate micro-layers, silt layers and fine-grained lenses, analyze soil samples for gradation and obtain grain-size curves. Site history must be evaluated to understand the mutual effect of existing subsurface conditions and grouting program on each other. Anomalies such as unexpected changes in drilling or grouting, as well as changes in effluent pH should be recorded.

Groundwater properties at the project location such as pH should be measured to determine the effectiveness of the selected grout and the post-construction effect of grout material on groundwater pH. Groundwater with high pH can be very destructive to sodium silicate-based grouts, preventing initial gel formation and/or causing grout degradation with time, whereas, soils with very low pH can be very destructive to Portland-based cement grouts. However, low pH groundwater conditions can accelerate setting of sodium silicate grouts, while preventing the setting of acrylamide or acrylate grouts and inhibiting cementitious reactions. The presence of organic materials in the ground or groundwater can also have a dramatic effect on the gel times and quality of chemical and cement grouts. Chemical analysis of groundwater is useful in this respect, but should not replace at least one series of grout mixing tests using groundwater samples from the project location in the grout mixture.

5.0 CONSTRUCTION SPECIFICATIONS AND QUALITY ASSURANCE

Development of specifications for grouting projects is described in this section.

5.1 Specification Development

Primary Reference: Specifications for Rock Mass Grouting (Bruce and Dreese 2010)

Specifications for grouting projects, in general, can be classified into two types – prescriptive and performance specifications. Prescriptive specifications consist of regulatory procedural, supervisory, monitoring, and material use rules, which must be strictly followed by the contractor. Prescriptive specifications promote low-bid situations and discourage innovative engineering approach to problems. Performance specifications require the contractor to achieve pre-defined performance criteria for the final product by allowing the modification of design, construction, and performance of project components.

Grouting projects involve several components for which specifications need to be developed, such as specialized equipment, materials, procedures, personnel, and pay items. Hence, specification development requires highly skilled and experienced contractors and efficient communication between contractors and owners. The tasks and responsibilities to be allocated prior to developing specifications for grouting are enumerated by Weaver and Bruce (2007) and are not presented here. The book also contains a table of detailed tasks for all items to be addressed and defined in the technical specification, the items being as listed below:

- Mobilization/Demobilization/General
- Drilling and Redrilling
- Special Flushing
- Water Pressure Testing
- Grouting
- Standby (Owner rights and personnel)

The contractor is responsible for preparing a detailed Method Statement (working plan) which is consistent with the specification details with a description of all phases of work. The specification type and details and the Method Statement govern the owner's supervisory and monitoring control of the project. Performance specifications provide contractors with an opportunity to present alternative approaches, materials, and/or equipment for performing the work in order to achieve the desired performance.

5.2 Bid Items

5.2.1 Methods of Estimating Quantities

- **Test Grouting**. For medium and large projects, probably the most reliable method for estimating quantities is to conduct a test-grouting program, preferably during the design stage. The site chosen for testing should be geologically representative of what was found during subsurface exploration; and the means, methods, and materials must be substantially those envisaged for the production work.
- **Evaluation of Subsurface Information**. The evaluation of the samples from the subsurface program, as well as the results of water pressure tests and other tests, is a fundamental part of the initial stages of preparing a grouting estimate. However, care should be exercised on grounds of site variability, and technical complexity.
- **"Unit Take" Estimates**. A method frequently used during preparation of detailed estimates for drilling and grouting programs is called the "unit take." In this procedure, the area to be grouted is divided into horizontal reaches and vertical zones of varying properties, based on site geology and in-situ test results. Estimates are made of the number of primary and split-spaced holes required to complete each area and zone.
- **Experience**. The local knowledge held by contractors or engineers is invaluable in providing a "reality check" on quantities derived by other methods.

5.2.2 Bid Items

Experience indicates that the following items should be included in any estimate or bid schedule for a drilling and grouting program.

5.2.2.1 Mobilization and Demobilization, Lump Sum

Drilling and grouting equipment must be assembled at the job site before a grouting program can be started and must be removed from the site when the work is completed, regardless of the amount of work actually performed. A separate pay item for these operations, therefore, should be included in the specifications; and the contractor will be guaranteed payment, regardless of whether work under the other items of the program is performed.

5.2.2.2 Environmental Protection, Lump Sum

A separate pay item may be included in the specifications. Environment protection is defined as the retention of the environment in its natural state to the greatest possible extent during project construction.

5.2.2.3 Drilling Grout Holes, Linear Foot Rock, Soil, Grout

A minimum diameter hole is generally specified. If different diameter holes are required by the contract, separate pay items should be provided. Separate pay items may also be warranted for the various depths or angles or where some of the drilling is to be done under special conditions, such as from a gallery or tunnel. If it becomes necessary through no fault of the contractor to drill the grout from a hole after set, a special payment provision for re-drilling should be provided (typically 50% of the rock drilling rate).

5.2.2.4 Pressure Washing and Pressure Testing

Preliminary washing of the grout hole usually is included for payment as a part of the drilling operations, and a separate pay item is not necessary. Pressure washing and testing are essential parts of the grouting program and, therefore, should be paid for as a separate item. Quantities of pressure washing and pressure testing ordinarily are measured for payment purposes in terms of units of time required to do the work. Pressure washing and pressure testing are closely related, and the operations performed are similar; therefore, payments for both operations may be combined in one pay item. Although the extent of pressure washing will depend on the conditions actually encountered, an approximation of the amount that will be required, as well as the amount of pressure testing expected to be done, should be made for inclusion in the estimate.

5.2.2.5 Grout Placement, by Volume or Pump Hour

The pay item for placing grout should cover the labor, the use of equipment, and the necessary supplies (other than grouting materials) required to mix and to inject the grout into the holes. Placing grout is frequently paid for by the volume of mixed grout and/or by the pump hour. An estimate of the quantity of grout must be made even though the actual amount is not known in advance. Payment for grout injection by the hour may be more appropriate in certain cases, and would include labor and use of equipment to inject the grout into the holes.

5.2.2.6 Connections to Grout Holes, Lump Sum or Per Connection

The labor required to hook up to a grout hole is independent of the effort involved in placing grout, and a separate payment may be desirable for each hookup or connection. The payment may consist of a fixed or bid price per grout hookup or connection.

5.2.2.7 Grout Materials, by Volume or Weight

Separate pay items should be established for each of the grout materials (except water) anticipated or planned to be used. The estimated quantity of each, expressed by volume or weight, should be derived from past experience, knowledge of the geologic conditions, and from test grouting, if performed. Clear distinction must be drawn with respect to the items being paid for under Grout Placement (above). The volume of grout placement must be consistent with the weights of the various grout materials.

5.2.2.8 Grout Injection Parameter Recording and Analysis

If a project warrants a high degree of real time parameter monitoring and analysis, then the computer-based system should be paid for separately, either as a lump sum or as a weekly or monthly recurrent fixed cost.

5.3 Specifications

Specifications for grouting projects must be tailored to achieve the specific objectives of the project, while exercising caution for "cut and paste" efforts. Factors such as performance requirements, site conditions, design specifications and economic considerations play an important role in developing specifications for grouting. No standard specifications exist that are directly applicable for all types of grouting projects. Hence, it is important to understand various specification items associated with different types of grouting. Guide specifications by ASCE and experienced grouting contractors are provided in this section.

5.3.1 Permeation Grouting Specifications

Guide specifications developed by Hayward Baker (Website: Hayward Baker) for permeation grouting are presented in this section. In order to develop performance specifications for permeation or chemical grouting projects, it might be necessary to include additional items or remove certain items to ensure that all project requirements are met.

1. General
    1.1. Introduction
    1.2. Intent
    1.3. Standards and References
    1.4. Definitions
    1.5. Scope of Work
    1.6. Submittals

1.7. Quality Assurance

2. Equipment and Materials

   2.1. Grouting Equipment

   2.2. Grout Pipes

   2.3. Grout Materials

3. Execution

   3.1. Site Examination

   3.2. Site Preparation

   3.3. Permeation Grouting

   3.4. Grouting Mixing Method

   3.5. Injection Procedures

   3.6. Field Quality Control

   3.7. Testing and Inspection

   3.8. Restrictions

4. Payment

   4.1. Method of Payment

5.3.2 Compaction Grouting Specifications

Guide specifications for compaction grouting developed by the ASCE are presented here (ASCE 2010).

1. Scope of work

2. Access and site conditions

3. Treatment area and depth

4. Subsurface pipes and utilities

5. Materials

   5.1. Grout mixture

   5.2. Cement

   5.3. Aggregates

   5.4. Water

6. Drilling and grouting equipment
    6.1. Drilling equipment
    6.2. Grout casing
    6.3. Casing withdrawal system
    6.4. Grout batcher/mixer
    6.5. Grout pump
    6.6. Grout delivery line
    6.7. Pressure gauges
7. Data acquisition and reporting
    7.1. Logged information
    7.2. Real-time computer monitoring
    7.3. Daily report
    7.4. Movement monitoring system
8. Communication system
9. Order of work
10. Drilling
    10.1. Establishing grout holes
    10.2. Hole location
    10.3. Control of drilling circulation flush
    10.4. Water injection
    10.5. Drilling log
11. Grout injection
    11.1. Depth confirmation
    11.2. Sequence
    11.3. Grout staging
    11.4. Access requirements
    11.5. Injection rate
    11.6. Grout refusal criteria
        11.6.1. Duration of pumping at maximum specified header pressure

11.6.2. Sustained pumping at maximum specified of header pressure

11.6.3. Limit allowable displacement of adjacent structure

11.6.4. Limit unwanted displacement of ground surface during grouting stage

11.6.5. Limit grout volume injected

11.7. Improperly grouted holes

11.8. Grout jacking

11.9. Hole completion

12. Site maintenance and restoration

12.1. Housekeeping

12.2. Site cleanup

13. Submittals

13.1. Grouting plan

13.2. Monitoring procedures

5.3.3 Jet Grouting Specifications

Guide specifications for jet grouting were developed by the ASCE (ASCE 2009).

1. General

1.1. Scope, Project Objectives and Job Site Conditions

1.2. References

1.3. Definitions

1.4. Qualifications

1.4.1. Project experience

1.4.2. Personnel experience

1.5. Submittals

1.5.1. Qualifications

1.5.2. Jet grouting equipment

1.5.3. Grout mix design

1.5.4. Field demonstration test program

1.5.5. Jet grouting procedure

1.5.6. QA/QC and verification procedures for field test and production work

1.5.7. Daily reports

2. Materials and Equipment

2.1. Materials: Cement, slag, fly ash, potable water, bentonite, material component ratios

2.2. Equipment: General equipment, drilling equipment, grout mixing and injection equipment, jet grouting pump, compressor, filling grout pump, jet grout tools, equipment instrumentation.

3. Execution

3.1. Test program

3.2. Production work

3.3. Quality control/quality assurance

3.4. Daily reports

3.5. Acceptance criteria

3.5.1. Accurate repetition of test program parameters

3.5.2. Minimum core recovery of 85%, subject to coring penetration rate, overall integrity and presence of gravel below jet-grouted soil

3.5.3. Permeability

3.5.4. Minimum 28-day compressive strength of jet-grout samples (grout only)

3.5.5. Minimum overlap thickness

3.5.6. Verticality and horizontal tolerances

4. Measurement and Payment

4.1. Measurement

4.1.1. Mobilization – measured as lump sum

4.1.2. Test program, including verification testing – measured as lump sum

4.1.3. Jet grouting – measured as lump sum

4.1.4. Coring, if used for verification testing – Linear foot per hole (ft./hole)

4.2. Payment

4.2.1. Mobilization – paid as lump sum

4.2.2. Test program – paid as lump sum

4.2.3. Jet grouting – paid as lump sum

4.2.4. Coring, if used for verification testing – Linear foot per hole (ft/hole)

5.3.4 Rock Grouting Specifications

Weaver and Bruce (2007) suggested the following items and related tasks to be addressed and defined in the technical specifications for rock fissure grouting.

5.3.4.1 Mobilization and Demobilization

Recommended payment method: Lump sum (typically 50-60% on mobilization, balance on demobilization)

- Number of project phases (i.e. interim moves)
- Project duration restraints
- Site location
- Facilities to be provided on site on arrival
- Facilities to be provided for use by other parties
- Site preparation (e.g., grout caps, access roads, scaffolding)

5.3.4.2 Drilling and Redrilling

Recommended payment method: Per linear foot (with a provision for a reduced redrilling rate for hardened grout)

- Hole quantities, location, length, orientation, inclination and number
- State length and method (e.g., upstage vs. downstage)
- Hole diameter (usually given as minimum)
- Unacceptable events during construction (e.g., air flush in rock)
- Special drilling method requirements (e.g., coring specific type of holes for investigation or verification)
- Deviation and straightness measurement and tolerances and measuring method and frequency
- Proposed course of action in extreme or unforeseen circumstances (e.g., major flush loss, rod drops)
- Requirements for logging, presentation and interpretation

- Environmental restrictions (handling of spoils, dust)
- Requirements for any standpipes or casing to be used
- Routine hole washing requirements

5.3.4.3 Special Flushing

Recommended payment method: Per crew hour

- Purpose and measures of success
- Duration and method of flushing
- Minimum and maximum pressures and flow rates
- Use of flushing aids
- Handling of spoils

5.3.4.4 Water-Pressure Testing

Recommended payment method: Per crew hour for multiple-pressure or extended tests; per test for simple, short (e.g., 5 to10 minute) tests

- Purpose and measures of success
- Pressure and flow limits
- Durations at each pressure
- Upstage vs. downstage
- Methods and accuracy of data recording, calculation, display and analysis
- Investigatory and verification testing requirements

5.3.4.5 Grouting

Recommended payment method: Per pump hour, per kilogram (or lbs) for materials mixes and possibly per month for specified levels of quality assurance and quality control monitoring, if not otherwise included.

- Stage length and method
- Primary, secondary, tertiary, etc. sequencing to closure
- Delays between grouting adjacent holes or phases

- Pressures and flow rates
- Refusal criteria
- Preconstruction laboratory or field testing requirements
- Routine quality assurance and quality control procedures and methods
- Accuracy of data recording, calculation, display and analysis
- Properties of various grout mixes and a plan to change them
- Procedures for unusual situations (e.g., runaway takes, zero takes, interconnections, surface leaks)
- Equipment details (including ancillaries such as packers and lines)
- Unacceptable methods (e.g., paddle mixers, w/c ratio > 2 by weight)
- Materials that can be used
- Hole backfilling requirements
- Relationship of drilling and permeability testing to grout takes
- Communication means

5.3.4.6 Standby

Recommended payment method: Per crew hour

- Circumstances under which the owner instructs
- Definition of crew size and composition

5.3.5 Void Filling Specifications

Void fill grouting currently lacks acceptable performance-based specifications for direct application to transportation projects (Website: http://geotechtools.org/). Method specifications that serve as a guide were developed by Healy and Head (1984) for bulk infill grouting of old mines. Both specifications provide guidance for bulk infill grouting projects and should be tailored for project site conditions. The guide specification provided in this section was obtained from PennDOT, developed for scour repair activity.

Guide Specification – Concrete Filled Forms for Scour Repair (Welsh 1997)

Description

This item should govern for the construction of concrete-filled containers for scour repair in accordance with these specifications and with the lines, grades, design, and dimensions shown on the plans or established by the Engineer.

Synthetic textile forms are employed as forms for concrete units. The units are pumped in place, and connecting dowels are used to ensure interlocking between the tubes or bags.

Forms

Containers (tubes or bags) for concrete placement should consist of a woven geotextile from stabilized yarns. Each container should be designed to remedy each particular scour zone when pumped with concrete, or in such a way that when a group is placed together, the scoured area is protected. These containers should be constructed with a minimum of one self-sealing valve to facilitate concrete pumping. If there is uncertainty in the scour void dimensions, tubes and bags should be field sewn to ensure that the height of the inflated concrete containers will not be more than one-half of the width.

The geotextile should meet the requirements listed in Table 8-13.

Table 8-13. PennDOT Minimum Required Properties for Polypropylene Geotextiles

Physical Property	Test Method	Unit	Values
Weight (Double Layer)	ASTM D3776	lbs./ft.	7.5
Thickness	ASTM D1777	in.	23
Mill Width		in.	80/165
Grab Tensile Strength	ASTM D4632	lb. ft./s^2 (lbf)	320 Warp – 300 Fill
Grab Tensile Elongation	ASTM D4632	Percent, %	18 Warp – 22 Fill
Burst Strength	ASTM D3786	psi	625
Trapezoidal Tear Strength	ASTM D4522	lbf	130 Warp – 130 Fill
Puncture Strength	ASTM D4833	lbf	80
Water Flow Rate	ASTM D4491	ft^3/s	105
Coefficient of Permeability	ASTM D4491	in./s	0.9
Permittivity (k/l)	ASTM D4491	1/sec.	1.5
Porosity	ASTM D737	$in.^3/min/in.^2$	300

Reinforced Dowel Rods (If Required)

Reinforcing dowels will be constructed of stainless steel or an approved equal. The type and strength of the rods should be submitted to the Engineer for prior approval. Rods should be embedded at least 0.3 m (1 ft.) into the lower bag or tube and protrude 0.3 m (1 ft.) into the upper bag or tube at each location. For tubes, these dowels should be spaced one meter apart on center.

5.3.6 Slabjacking Specifications

The guide specification given in UFGS-32 01 29.62 (USACE 2008) covers the requirements for slabjacking rigid pavements for roads, streets, parking areas, airfields, and other general applications. The specifications may be edited by adding, deleting, or revising the text provided in the guide specification as per the project requirements.

1. General
   1.1. Unit Prices
      1.1.1. Measurement
         1.1.1.1. Quantity of Portland cement grout

1.1.1.2. Quantity of Portland cement

1.1.1.3. Number of Holes

1.1.1.4. Broken Slabs

1.1.2. Payment

1.1.2.1. Portland Cement Unit Price

1.1.2.2. Drilled Holes

1.2. References

1.3. Submittals

1.4. Quality Assurance

1.4.1. Bench Marks

1.4.2. Testing Facilities

1.4.3. Cement

1.4.4. Aggregate

1.5. Delivery, Storage, and Handling

1.5.1. Provisions for Cement

1.5.2. Provisions for Aggregates

1.6. Environmental Requirements

2. Products

2.1. Executing Equipment

2.1.1. Grout Plant

2.1.2. Water Tanker

2.1.3. Drilling

2.1.4. Flow Cone

2.1.5. Miscellaneous

2.2. Grout Mixture

2.3. Mineral Aggregate

2.3.1. Particle Shape

2.3.2. Grading

2.3.3. Deleterious Materials

2.4. Pozzolans and Fly Ash

2.5. Portland Cement

2.6. Water

2.7. Chemical Admixtures

2.8. Proportioning of Materials

2.9. Tests, Inspections and Verifications

2.9.1. Daily Report

2.9.2. Compressive Strength

2.9.3. Expansion

2.9.4. Set Time

2.9.5. Fluidity

3. Execution

3.1. Pavement Inspection

3.2. Drilling Holes for Grout Injection

3.3. Wash Holes

3.4. Jacking

3.5. Raising of Slabs

3.6. Sealing of Injection Holes

3.7. Plan Grade Requirements

3.8. Replacing And Repair of Damaged Pavement

3.9. Production Sampling and Testing

3.9.1. Aggregates

3.9.2. Field Test Specimens

3.10. Protection of Pavement

3.11. Acceptance of Work

5.4 Inspection Control and Verification

Void filling and slabjacking problems tend to necessitate "one-of-a-kind" grouting solutions, which makes Guide Specification difficult. It is suggested that the Engineer developing the specification and construction control use the preceding specifications as a guide. Also, the

bidding method will depend on the amount of knowledge available on the problem; information on the problem and its potential solution will determine the actual bidding methods and risks to be placed on the grouting Contractor. This can range from the cost-plus through a lump-sum-method.

5.5 Quality Assurance

Effective quality assurance procedures based on well-developed testing and performance criteria are essential to ensure the success of a grouting program. Every drill hole is a potential source of information, and drill logs and test results obtained manually or electronically provide valuable data for developing quality assurance procedures. The key is that the data are studied in real time or very soon thereafter, and that any adjustments or changes to the grouting program can be effected in a timely routine and responsive fashion.

Similarly, the grouting data provide equally valuable information of how the ground is behaving in response to the treatment. Close examination of grout pressure/volume/time records, again manually or electronically recorded and/or displayed, will provide vital insight into the effectiveness of the operation to that point. For example, if a rock grouting operation is progressing well, then the higher order holes will have smaller grout takes and will need slower rates of injection at equivalent pressures to attain refusal than the primaries.

During grouting, it is essential to frequently and routinely monitor the fluid properties of the materials being injected. Thus, for rock fissure grouting or soil permeation grouting, it is instructive to routinely record the fluidity, the specific gravity, the setting time, and the stability, whereas for compaction grouting, only slump testing may be of relevance.

As a further general point, it may be emphasized that the site's geotechnical situation must be "baselined" prior to grouting. This means that the key virgin parameters must be measured (such as density or permeability), depending on the nature of the project. Following the monitored execution of the grouting work, verification testing must be conducted to demonstrate the effectiveness of that work. The nature of the testing must reflect the goals of the project.

Finally, grouting lends itself, and indeed has a great need for preconstruction test programs. These permit the designer's assumptions and the contractor's methods to be tried, tested, and verified prior to the commencement of the production works. This is often overlooked, and is aimed at enhancing quality and reducing problems, technical and contractual.

5.6 Instrumentation Monitoring and Construction Control

Monitoring the instrumentation for accuracy and sufficiency of field-collected data is essential for the success of any grouting project. The required level of monitoring and responsibilities are typically included as Quality Control items in the specifications. Since different grouting techniques require different types of equipment, design, and testing methods, monitoring activities should be developed specifically for each project.

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Chapter 9

PAVEMENT SUPPORT STABILIZATION TECHNOLOGIES

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1.0 INTRODUCTION

This chapter provides a review of subgrade stabilization methods for foundation support of pavement structural sections (i.e., surface asphalt or concrete pavement, base and subbase layers). Proper treatment of problem soil conditions and the preparation of the foundation for pavement construction are extremely important to ensure a long-lasting pavement structure that does not require excessive maintenance. When compaction alone will not improve the subgrade soil or may even make the conditions worse, the conventional technique is to remove and replace poor subgrade soils with better, compacted materials (i.e., a form of mechanical stabilization). However, this is not always the most economical or even desirable treatment as excavation may create disturbance and problems related to removal and disposal. Ground modification using other stabilization technologies can often be more effective alternatives to improve the strength and modulus for both pavement construction and performance. These technologies fall into one of three categories.

1. Mechanical stabilization using: thick granular layers; blending with more competent materials; geosynthetics (e.g., geotextiles and geogrids) in conjunction with granular layers; or lightweight fill materials.
2. Admixture stabilization of weak soils by using chemical agents (i.e., lime, cement, asphalt, fly ash or other admixtures.
3. Stabilization through moisture control (i.e., dewatering, drainage /or moisture barrier).

A number of specific technologies can be used within each of these categories, the most common of which are listed in Tables 9-1, 9-2, and 9-3. Alternate stabilization methods for pavement construction and support will be reviewed in this chapter, and guidance will be provided for the selection of an appropriate method.

Table 9-1. Subgrade Stabilization Methods for Pavement Support – Mechanical Category

Stabilization Method	Soil Type	Improvement	Remarks
Thicker gravel	Silts and clays	None	Reduces dynamic stress level
Blending	Moderately plastic Low or non-plastic	None Improved gradation Reduced plasticity Reduced breakage	Too difficult to mix
Geosynthetics	Silts and clays	Minimize disturbance Strength gain through consolidation	Fast, plus provides long-term separation
Lightweight fill	Very weak silts, clays, peats	None Thermal barrier for frost protection	Fast, and reduces dynamic stress level
Recycled material	Silts and clays	None	To replace or blend with gravel; may be lightweight

Source: Rollings and Rollings 1996

Table 9-2. Subgrade Stabilization Methods for Pavement Support – Admixture/Additive Category

Stabilization Method	Soil Type	Improvement	Remarks
Portland cement	High plasticity Coarse-grained		Less pronounced hydration of cement with high plasticity soils Hydration of cement with coarse-grained soils
Lime	High plasticity	Drying Strength gain Reduced plasticity Coarser texture Long-term pozzolanic cementing	Rapid for all improvements, except for long-term pozzolanic cementing, which is slow
Lime	Coarse with fines	Same as for high plastic soils	Dependent on quantity of plastic fines
Lime	Non-plastic	None	No reactive material
Lime-fly ash	Same as lime	Same as lime	Covers broader range than lime
Lime-cement-fly ash	Same as lime	Same as lime	Covers broader range than lime or cement alone
Bituminous	Coarse	Strengthen/bind waterproof	Asphalt cement or liquid asphalt
Bituminous	Coarse with fines	Same as coarse	Liquid asphalt
Bituminous	Fines	None	Cannot mix
Pozzolanic and slags	Silts and coarse	Acts as a filler Cementing of grains	Dense and strong Slower than cement
Other chemical admixtures	Plastic	Strength increase and volume stability	See vendor literature Difficult to mix

Source: Rollings and Rollings 1996

Table 9-3. Subgrade Stabilization Methods for Pavement Support – Moisture Control Category

Stabilization Method	Soil Type	Improvement	Remarks
Improved drainage	Sand, silt and wet, low plastic clays	Decreased moisture, increased strength	Requires time and grade for runoff
Partial and full encapsulation (e.g., with geomembranes)	Plastic and collapsible	Reduced change in moisture	Long-term moisture migration problem

Source: Rollings and Rollings 1996

1.1 Description

In this chapter, a description of the each category of pavement support stabilization methods will be presented along with the specific technologies that fall under this category. The most important consideration for any stabilization technique used for pavement support is the provision for a uniform soil relative to its stiffness. Generally, the stiffness (in terms of resilient modulus) of some soils is highly dependent on moisture and stress state. This uniformity can be achieved through soil sub-cutting (i.e., excavating) and recompacting the subgrade soil or other stabilization techniques. In some cases, the subgrade soil can be treated with various admixtures/additives to improve the strength and stiffness characteristics of the soil. Stabilization may also be used to improve soil workability (i.e., improve compactability), minimize strength and modulus loss due to moisture, reduce swelling and shrinkage of expansive materials, and/or mitigate problems associated with frost heave.

Stabilization of subgrade soils is usually performed for three reasons:

1. To create a construction platform over or with (e.g., by drying) wet, weak soils, facilitating placement and compaction of the pavement section layers. For this case, the *stabilized* subgrade soil is not usually considered as a structural layer in the pavement design process. However, guidance is provided in this chapter for assessing long-term improvement in pavement support provided by stabilized subgrades that can be considered in the pavement design.
2. To strengthen a weak soil and restrict the volume change potential of a highly plastic or compressible soil. For this case, the stabilized soil is considered "*modified*" and is usually given some structural value or credit in the pavement design process.
3. To reduce moisture susceptibility of fine-grained soils.

The methods of modification and/or improvement and type of soil for each of the technologies are listed by stabilization category in Tables 9-1, 9-2, and 9-3.

1.1.1 Mechanical Stabilization

Mechanical stabilization is the replacement of a particular thickness of weak subgrade material with a more competent, compacted material. Of course, if compaction alone provides suitable stiffness and uniformity, other stabilization methods will not be required. Where materials must be replaced, granular backfill (i.e., sand, gravel, and/or recycled materials such as crushed concrete or recycled asphalt) is often used. The effectiveness of the compacted granular material can be enhanced by using geosynthetics (geotextiles, geogrids, geocomposites, or geocells). Less competent finer grained subgrade soils can be blended with granular material to improve performance. Lightweight materials can also be used to replace

subgrade materials to reduce the vertical stress to the underlying subgrade. Thus mechanical stabilization includes use of compacted granular layers, blending soils with granular layers, use of geosynthetics with granular layers, the use of lightweight fill materials or recycled materials in lieu of or in combination with granular layers as indicated in Table 9-1.

1.1.1.1 Thick Granular Layers and Blending

Thick granular layers, often a form of over excavation and replacement, provide a working platform for construction equipment to move over the section to be paved without disturbing the underlying subgrade. The granular layer must be sufficiently thick to spread the load such that the stress levels at the subgrade do not exceed its bearing capacity, which would result in rutting of both the subgrade and granular layer. Thick granular layers are also used to avoid or reduce frost problems by providing a protection to the underlying subgrade layers. The increase in gravel thickness (minus an allowance for rutting) can also contribute to the support of the pavement; however, this increased thickness is usually not considered in the design as the layer may become contaminated over time due to intermixing with the underlying subgrade soils. Contamination can occur due to migration of fines (minus No. 200 sieve) up into the granular layer either due to pumping or dynamic mechanical action, which may reduce the strength, stiffness and drainage characteristics of the granular layer over time.

Blending with granular materials such as gravel and, more recently, recycled pavement material with poorer quality soils also can provide a working platform. The granular material acts to create a drier condition and tends to reduce the influence of plasticity. This method may be cost effective where limited sources of granular material are available on the project.

1.1.1.2 Geosynthetic Separation

A geosynthetic layer (typically a geotextile) can be placed beneath granular layers (either base/subbase or working platform (e.g., thick granular layers)) to prevent intermixing with the adjacent subgrade soils, thus maintaining the thickness and integrity of the granular layer over the life of the pavement. Separation is especially critical for open graded granular layers used to enhance drainage. Open graded coarse (e.g., 3–inch) angular aggregate placed over a separation geotextile also provides a very good free draining layer for initial stabilization of extremely soft, wet subgrades. For wet subgrade soils, the geotextile layers will also have to satisfy filtration criteria. This relatively low cost stabilization method should be considered for any subgrade soils containing significant amount of fines (e.g., USCS: SC, CL, CH, ML, MH, OL, OH, and PT or AASHTO: A-5, A-6, A-7-5, and A-7-6 type soils).

1.1.1.3 Geosynthetic Stabilization

Where the soils are normally too weak to support the initial construction work, stronger geosynthetics (i.e., stronger geotextiles than required for separation, geogrids, geocomposites, or geocells) can be used to reinforce the soils to both reduce the thickness of gravel required to provide a working platform and improve the short and long-term performance of the roadbed support condition. In this application, separation must also be provided and the geotextile, or granular layer in the case of geogrids or geocells, must also meet filtration requirements for the underlying subgrade, to allow water in the subgrade to freely drain to rapidly reduce pore water pressure build-up under dynamic loads. Excess pore water pressure could further weaken the subgrade soil and may even result in a waterbed effect inhibiting compaction of the granular layer. When these hydraulic requirements are met, geosynthetics used with gravel have been found to allow for subgrade strength gain over time. As the geosynthetic provides multiple functions, which both benefit construction and allow for subgrade improvement with time, AASHTO (2014a) has identified geosynthetic applications where the subgrade undrained shear strength is less than about 2000 psf (California Bearing Ratio, CBR, about 3) as mechanical stabilization.

1.1.1.4 Lightweight Fill

Lightweight fill materials have been used to replace granular soils for stabilization of subgrades by providing similar benefits of gravel while reducing stresses applied to the subgrade due to the material weight, especially when there is a concern for settlement of pavements constructed on highly compressible soils (e.g., marsh land). This is particularly the case for deeper deposits of weak soft subgrade soils where shallow surface stabilization may not be effective and thicker granular aggregate, as discussed in Section 1.1.1.1 of this chapter, may be effective for control of deformation under wheel loading, but would increase the settlement. In many cases, the use of lighter weight materials on soft soils will likely result in both reduced settlement and increased stability. The Lightweight Fill Chapter reported a wide range of lightweight fill materials (e.g., see Table 3-1 in Chapter 3), many of which have been used for creating a stabilized working platform for constructing pavement sections (e.g., geofoam, foamed concrete, tire shreds, expanded shale and clay, fly ash, bottom ash, boiler slag, and air-cooled slag).

1.1.1.5 Recycled Materials

Recycling, in principle, is a very powerful and often political concept. While the benefits of recycling include conservation of aggregate and preservation of the environment, it requires serious consideration. The long-term performance of recycled materials in pavements and, in some cases the environmental impact, must be carefully evaluated to avoid costly

performance and maintenance issues. There are two forms of recycling for pavement support: 1) reuse of the pavement materials themselves, and 2) the use of recycled waste materials for subgrade stabilization or as a substitute for aggregate.

Onsite reuse of pavement materials includes recycling of the asphalt or concrete pavement in the subgrade stabilization layer, the construction of new pavements, and/or rehabilitating existing pavements. The method of recycling the pavement will, in most cases, depend on whether the pavement has an asphalt concrete (AC) or Portland cement concrete (PCC) surface pavement. In either case, the material could be rubblized, or, in some cases, processed (e.g., sieving, stockpiling, and reusing the reclaimed asphalt pavement (RAP) materials or recycled concrete materials (RCM) plus the aggregate base). Both pavement types can also be rubblized in-place and compacted. This procedure is known as rubblize and roll for PCC pavements and full depth reclamation for AC pavements. For AC pavement materials, there are also several other methods, including hot mix asphalt recycling, hot in-place recycling, and cold in-place recycling, all of which produce a bound product, which is beyond the scope of this manual.

The second type of recycling, recycled waste materials, involves a number of materials that have been used in permanent construction, practically all of which were covered in the previous Chapter 3 Lightweight Fill, since they have a lighter weight than conventional aggregate.

1.1.2 Admixture (a.k.a. Chemical) Stabilization

Stabilization with admixtures involves mixing the soil with lime, cement, fly-ash, viscoelastic materials such as bitumen, and proprietary chemical stabilizers. Proprietary stabilizers will not be covered in this chapter, but of course the vendors will provide product and performance information, which the user must evaluate before using. Admixture stabilization is used for controlling (and/or mitigating) the swelling and frost heave of soils and improving the strength characteristics of unsuitable soils. The effectiveness of the selected admixture will depend on the type and amount of additive and the soil to be treated. These improvements arise from several important mechanisms including filling or partially filling voids between soil particles thus reducing its permeability to protect underlying moisture sensitive layers and coating of particle surfaces by the additive to limit the moisture sensitivity. The admixture stabilizing agent also acts by binding the particles of soil together, adding cohesive shear strength and increasing the difficulty with which particles can move into a denser packing under load. Particle binding serves to reduce swelling by resisting the tendency of particles to move apart. The particles may be bound together by the action of the stabilizing agent itself (as in the case of asphalt cement), or may be cemented by chemical reaction between the soil and stabilizing agent (as in the case of lime or Portland cement).

Additional improvement can arise from other reactions that affect the soil fabric (typically by flocculation) or the soil chemistry (typically by cation exchange).

1.1.2.1 Cement (Portland Cement and Cement Kiln Dust)

Portland cement is widely used for stabilizing low-plasticity clays, sandy soils, and granular soils to improve the engineering properties (strength and stiffness). Increasing the cement content increases the quality of the mixture, although to high could induce shrinkage cracking. At low cement contents, the product is generally termed cement-modified soil. A cement-modified soil has improved properties of reduced plasticity or expansive characteristics and reduced frost susceptibility. At higher cement contents, the end product is termed soil-cement or cement-treated base, subbase, or subgrade.

Cement kiln dust (CKD) is an industrial by-product with constituents including partially calcined and unreacted raw feed, clinker dust, and fuel ash, which is enriched with alkali sulfates, halides and other volatiles. For the purpose of soil stabilization, it can be segregated into two categories, pre-calciner kiln dust and long-wet or long-dry kiln dust. The generally coarser pre-calciner kiln dust contains higher free lime and concentrated with alkali volatiles, while the dust from long kilns contains more calcium carbonate and is limited in free lime (Parsons et al. 2004). As the pre-calciner CKD contains substantial amounts of free lime, it can be expected to perform more like lime.

1.1.2.2 Lime

Lime or pozzolanic stabilization of soils improves the strength characteristics and changes the chemical composition of some soils. The strength of fine-grained soils can be significantly improved with lime stabilization, while the strength of coarse-grained soils is usually moderately improved. Lime has been found most effective in improving workability and reducing the swell potential with highly plastic clay soils containing montmorillonite, illite, and kaolinite. Lime is also used to reduce the water content of wet soils during field compaction. In treating certain soils with lime, some produced soils are subject to micro cracking due to shrinkage. Hydrated lime, in powder form or mixed with water as a slurry, is used most often for stabilization.

1.1.2.3 Fly Ash

Fly ash, also termed coal ash, is a mineral residual from the combustion of pulverized coal. It contains silicon and aluminum compounds Fly ash is classified according to the type of coal from which the ash was derived. Class C fly ash is derived from the burning of lignite or subbituminous coal and is often referred to as “high lime” ash because it contains a high percentage of lime. Class C fly ash is self-reactive or cementitious in the presence of water,

in addition to being pozzolanic. Class F fly ash is derived from the burning of anthracite or bituminous coal and is sometimes referred to as "low lime" ash. It requires the addition of lime to form a pozzolanic reaction. To be of acceptable quality, fly ash used for stabilization must meet the requirements indicated in ASTM (2014) C593, *Standard Specification for Fly Ash and Other Pozzolans for Use with Lime for Soil Stabilization*.

1.1.2.4 Asphalt

Asphalt-stabilized soils are generally used for base and subbase construction, but can also be used to stabilize loose sand type subgrades and to reduce moisture effects in cohesive soils. Use of asphalt as a stabilizing agent produces different effects, depending on the soil type and properties, and may be divided into three major groups: 1) sand-bitumen, which produces strength in cohesionless soils, such as clean sands, or acts as a binder or cementing agent, 2) soil-bitumen, which regulates and maintains the moisture content of cohesive fine-grained soils, and 3) sand-gravel bitumen, which provides cohesive strength to pit-run gravelly soils with inherent frictional strength. The durability of bitumen-stabilized mixtures generally can be assessed by measurement of their water absorption characteristics. Treatment of soils containing fines in excess of 20% is not recommended.

1.1.2.5 Combinations

Stabilization of coarse grained soils having little or no fines can often be accomplished by the use of lime-fly ash (LF) or lime-cement-fly ash (LCF) combinations that, when mixed with lime and water, form a hardened cementitious mass capable of obtaining high compressive strengths. Lime and fly ash in combination can often be used successfully in stabilizing granular materials, since the fly ash provides an agent with which the lime can react. Thus, LF or LCF stabilization is often appropriate for base and subbase course materials.

1.1.3 Moisture Control

Moisture control should always be incorporated into pavement design; however, controlling moisture prior to and during construction may also be required to stabilize moisture sensitive soils to support construction activity. Moisture control can take the classical form of dewatering using conventional trench drains or horizontal blanket drains, the latter of which can be used as a cap layer for capillary rise in fine grained soils and correspondingly frost heave in cold regions. Moisture can also be controlled by preventing moisture changes, either wetting or drying, through encapsulation.

1.1.3.1 Improved Drainage – Dewatering

Significant subgrade stability improvement can be obtained by adding underdrains on new construction projects and by cleaning out the existing underdrain outlets on rehabilitation projects. Once the underdrain systems are in-place and functioning, the drainage system can typically reduce subgrade pumping problems within a few days, but may take longer depending on the characteristics of the in situ materials. Soils that are subject to densification and are not free draining (percent saturation exceeding 80 – 90%) within 2 to 4 feet of the surface are not expected to support construction traffic. This level of saturation is frequently observed to be the limit for compaction stability when developing moisture-density curves in the laboratory. Saturated soils with more than 10% fines are not expected to be drainable with respect to supporting construction traffic. Moisture reduction by evaporation (e.g., disking and aeration) may be more feasible than gravity drainage for these types of soils. Cutoff drains (i.e., deeper trenches outside of the roadway) can also be used to prevent groundwater from entering the roadway section.

1.1.3.2 Geosynthetic Drainage

Prefabricated geocomposite drains are used to replace or support conventional drainage systems. During the past 20 years or so, a large number of geocomposites drainage products have been developed, which consist of cores to convey water and geotextiles on one or both sides for filtration. A variety of cores are used and include extruded and fluted plastics sheets, three-dimensional meshes and mats, plastic waffles, and nets and channels. Some geotextiles alone (i.e., thick needle-punched nonwoven geotextiles) can provide some in-plane drainage and be used to dissipate pore water pressure or handle minor seepage problems.

As previously indicated, geotextiles can be placed beneath free draining (i.e., open graded) stabilization layers as a separator/filter to maintain drainage and improve the roadbed performance. However, where free draining aggregate is not used, not readily available, and/or relatively expensive, geosynthetics with lateral drainage capability (transmission function) may be placed at the subgrade-stabilization layer interface to: (i) provide drainage of the subgrade when a poorly draining, dense graded aggregate is used, and/or (ii) promote quicker drainage of the aggregate. For the first case of poorly draining aggregates, where water in a wet subgrade cannot readily drain upward into the stabilization aggregate, geosynthetics that allow some drainage in its plane (i.e., thick nonwoven geotextiles or geocomposites) would also allow for pore water pressure dissipation. For the second case, drainage geocomposites with sufficient compressive strength to support traffic without excessive deformation and with adequate flow capacity can be used to enhance drainage of the roadway system and provide excellent support for thinner stabilization gravel layers.

1.1.3.3 Partial and Full Encapsulation

Soil encapsulation is a foundation improvement technique that has been used to protect moisture sensitive soils from large variations in moisture content. The concept of soil encapsulation is to keep the fine-grained soils at or slightly below optimum moisture content, where the strength of these soils can support heavier trucks and traffic. This technique has been used by a number of states (e.g., Texas and Wyoming) on selected projects to improve the foundations on high volume roadways. It is more commonly used as a technique in Europe and in foundation or subbase layers for low-volume roadways, where the import of higher quality paving materials is restricted from a cost standpoint. By 1998, more than 100 projects had been identified around the world, usually reporting success in controlling expansive soils (Steinberg 1998).

1.1.4 Alternate Technologies

Geogrids and geotextiles can also be used to reinforce the base course of flexible pavement to improve its serviceability. Base reinforcement is especially viable for thin pavement systems used in secondary roads and the geogrid base course layer may also serve to allow construction over moderate subgrade soils, mitigating construction disturbance issues.

Geocells have also been used to a limited extent for improving pavement support, especially when using lower quality aggregates, sand and recycled materials. Geocells are a 3-dimensional interconnected honeycomb type of geosynthetic used to confine granular materials and are most often associated with erosion control applications. They were actually developed in cooperation with the US Army Corps of Engineers for roadway stabilization to allow deploying military equipment on beach fronts (Webster and Watkins 1977, Webster 1979 and Webster 1981). Cost has generally been the main determent of their use in roadway stabilization applications; however, the increased cost of pavement materials has made their use more viable in pavement construction. Their ability to confine granular base materials, and increase in situ soil stiffness and strength properties, allows for a reduced required base thicknesses and increased pavement service life. There are many more studies on geocell for roadway applications in recent years (e.g. Yang et al. 2013, Thakur et al., 2012, and Han et al. 2011).

Bio-treatment for subgrade stabilization in the past has had a mixed track record; however, new advancements have made this a more viable alternative. Bio-stabilization involves chemical and biological reactions which can be used to make unstable soils sufficiently strong and durable for pavement construction. The microbial process is more complex than the admixture technologies previously reviewed in this section and depends on many environmental factors such as temperature, pH, concentrations of donors, and acceptors of

electrons, concentrations and diffusion rates of nutrients and metabolites. Additional information on bio-treatment technologies can be found in DeJong et al. (2010), Mitchell and Santamarina (2005), and Ivanov and Chu (2008).

For existing pavements, disruption of traffic to completely reconstruct the pavement section is always a significant issue, even when it is desperately in need of repair. Two interesting, yet underused methods, electro osmosis and vacuum consolidation, provide an alternative stabilization method where disruption of traffic is not possible. Both methods are used to remove moisture for underlying pavement systems. Electro osmosis is the process of inducing pore water movement in capillaries under the influence of electric potential. The effective stress and shear strength of in situ soils is improved immediately so that the stability of treated soil can be improved. This technique is applicable to silts and clays with low permeabilities. Advantages include simple equipment, minimum surveillance and monitoring, and low mobilization cost.

Another stabilization method for weak silt and clay type subgrade issues, that may not require complete removal of pavement, is vacuum consolidation. In this method, the surface pavement is completely sealed (i.e., with a bitumen coated geotextile overlay fabric or a removable geomembrane). The seal is extended into a vertical trench at the edge of the road into the unsuitable material (e.g., the trench as constructed may later be used for an edgedrain). A vacuum is then applied under the pavement through the trench and base course layer. The vacuum removes excess moisture and consolidates the underlying soil, improving the stiffness of the subgrade layer.

Additional information on each of the technologies identified in this section can be found in *GeoTechTools*.

1.2 Historical Overview

Stabilization methods have been used for construction of roadways for thousands of years. The Romans used thick gravel layers, reed mats as separators to improve the performance of the gravel (i.e., similar to using geosynthetics) and drainage (a key feature) in most of their roads, a number of which still exist today. In the United States, corduroy roads (timbers roped or chained together) were often used to construct roads over soft ground in the 1800s.

The use of admixtures for soil stabilization began during the 1960s as an alternative to hauled-in aggregates due to general shortages of aggregates and petroleum resources. These methods fell out of favor by the 1980s, often due to faulty application techniques and inadequate design. However, over the past two decades, research, improved materials, and

better equipment have significantly improved both the performance and acceptance of these technologies.

Some lightweight fill materials have also been used for improved support of road base materials for decades, while others are relatively recent developments. Wood fiber has been used for many years by timber companies for roadways crossing peat bogs and low-lying land, as well as for repair of slide zones. The steel-making companies have produced slag since the start of the iron and steel making industry. Initially, the slag was stockpiled as waste materials, but beginning around 1950, the slag was crushed, graded, and sold for fill materials.

Geosynthetic type materials were actually first used in the 1920s and 1930s. Modern polymeric geosynthetics were first used for road construction in the 1960s, and design methods were first advanced by the United States Forest Service (USFS) for stabilization of logging haul roads in the early 1970s. Since that time, a number of advancements have been made in the materials (e.g., geogrids and geocomposites) and design methods. Monitoring of project performance over the past several decades has clearly shown that the performance benefits of using geosynthetics extend beyond the initial short-term stabilization performance to long-term roadway performance improvements.

One of the more recent significant changes in the use of stabilization methods is the improvement of pavement design model using the AASHTO (2008) Mechanistic Empirical Pavement Design Guide (MEPDG). Although not fully developed for stabilization alternatives, this design method has the potential for the inclusion of the stabilization layer in the design model, and using calibrated degradation factors for the stabilization lift would provide a direct method for evaluating the benefits of a stabilization layer on the long-term pavement performance. Calibration information is currently being developed by several agencies (e.g., Iowa DOT) that rigorously employ stabilization technologies.

1.3 Comparison/Applicability of Stabilization Technologies

For determining the applicability of different stabilization technologies, the largest geotechnical challenge is identifying their need in the first place and having adequate information to determine which method(s) are most suitable. Often these conditions are not clearly identified in the design stage, rather they are often discovered during the construction process, when it is too late and often too costly to delay construction and allow time to run the tests required to identify the ideal solution(s). A good subsurface exploration program that will clearly identify subgrade conditions along a roadway alignment that require stabilization and adequately characterize the subsurface conditions that are key to determining the applicable method(s) that should be considered. Guidance is presented in

FHWA 2010. In particular, geophysics should be considered to facilitate locating areas requiring stabilization along the roadway.

With adequate site information, the previous Tables 9-1, 9-2, and 9-3 and the following Table 9-4 can be used to help determine the applicability and suitability of each method.

Several state agencies have flow charts to assist in this process. However, usually these flow charts are restricted to a specific type of stabilization (e.g., chemical methods) and do not cover the complete range of methods identified in Tables 9-1, 9-2, and 9-3. In addition to the soil type applicability and anticipated improvement and remarks noted in Tables 9-1, 9-2 and 9-3, the relative benefits provided by each technology to support decisions on the selection of specific technologies to evaluate farther for a given application are listed in Table 9-4.

In addition to the comparisons in Table 9-4, users should consider that mechanical stabilization works immediately as it does not require a curing period. Geosynthetics used with gravel to stabilize fine grained subgrades provide long-term improvement in pavement performance by maintaining the integrity of the gravel over the life of the pavement and potentially long-term strength gains in the subgrade, albeit long term benefits are difficult to quantify. Mechanical stabilization with free draining granular material and geosynthetics improves subsurface drainage and provides frost protection (i.e., either improving uniformity during heave or eliminating frost if extended below the freezing front). Potential disadvantages of mechanical stabilization are that some excavation and disposal of materials is often required and adequate quality granular material must be available.

Admixtures provide an in-place improvement in the strength, modulus and durability of subgrade materials and work well with most soils. Admixtures mitigate heave from swelling or frost in certain admixture/soil combinations. The down side of admixtures is that they require up front lab testing to confirm their performance, good field control to obtain a uniform, long lasting product, and a required curing time of three to seven days. Long-term benefits have been difficult to quantify. There are also issues of dust control and weather dependency with some methods that should be carefully considered in the selection process. Admixtures do not work on all soil conditions and caution is especially advised when soils contain organics or sulfates.

Drainage should always be the first technique to apply/try and considered for use in conjunction with any of the other stabilization methods. Drainage, especially using underdrains, is the least costly technique to consider. However, some soils drain very slowly, if at all. Fully encapsulation can mitigate problems with swelling soils. Partial encapsulation does not require excavation; however, partial encapsulation is not a permanent solution and the longevity for performance is difficult to estimate.

Table 9-4. Benefits of Different Stabilization Methods

Stabilization Method	Increased Support	Uniform Subgrade Support	Improve Drainage	Frost Protection	Reduce Subgrade Rutting Potential	Reduce Pumping/Erosion - PCC	Materials & Equipment Readily Available	Prevent Base/Subgrade Mixing	Treats Highly Plastic Soils	Treats Granular Soils	Treats Silts/Fine Sands	Treats Low PI Clays	Reduce Moisture	Reduce Swell	Potential Dust Problem	Freeze-Thaw Deterioration	Leaching over Time	Design Procedure	Cost	Increased Excavation/Subcutting
More Gravel – Thicker Section	X	X	?	X	X	X	X		X	X	X	X								X
Separator – Geotextiles & Geogrids		X	X		X	X	X	X	X		X	X	X							
Reinforcement – Geogrids & Geotextiles	X	X			X		X	X	X		X	X						X	?	
Lightweight Fill		X		X					X		X	X								X
Lime Treatment < 3%		X		X	X	X		X	X				X	X	X	X	X			
Lime Stabilization	X	X		X	X	X		X	X				X	X	X	X	X			
Cement Treatment		X		X	X	X		X		X	X	X	X		X		X			
Cement Stabilization	X	X		X	X	X		X		X	X	X	X		X		X			
Lime-Fly Ash Stabilization		X			X	X		X		X			X		X		X			
LCF Stabilization		X		X	X	X		X		X			X		X		X			
Asphalt Stabilization	X	X	X		X	X	X	X	X										?	
Mechanical Stabilization – Compaction	X	X			X		X			X	X		X							X

X=A benefit of the respective stabilization method
?=A possible benefit of the respective stabilization method

1.4 Focus and Scope

The purpose of this chapter is to present an overview of the more common methods of stabilizing subgrade soils for pavement construction and improving the design life of pavement systems constructed on marginal soil conditions. The suitability of various stabilization technologies for different soil conditions, the anticipated improvement that should be anticipated for both construction and over the life of the pavement as well as the advantages, disadvantages and limitations of each technology will be reviewed. Typical geotechnical engineering parameters that are important to the design of each stabilization method are provided. In addition, design and construction considerations for effective application of stabilization technologies and those unique to each specific technology are presented.

The technical summary also presents guidelines and references for preparation of specifications along with suggested quality assurance procedures, including construction quality control. Cost information is provided to assist in the cost estimation and cost comparison of technologies that would be suitable for specific project conditions.

1.5 Glossary

A variety of terms are used with these various subgrade stabilization methods. For clarity, they are defined throughout this chapter, as listed below. The acronyms associated with terms, when applicable, are noted with the term.

Base course (base) – The layer or layers of specified or select material of designed thickness placed on a subbase or subgrade to support a surface course. The base course and subbase are part of the pavement structure and are usually crushed stone, crushed slag, crushed gravel and sand or combinations of these materials. (FHWA 2010)

Bio–treatment for subgrade stabilization – a bio-mediated soil improvement system as a chemical reaction network that is managed and controlled within the soil through biological activity whose byproducts alter the engineering properties of soil.

California Bearing Ratio (CBR) – the ratio of (1) the force per unit area required to penetrate a soil mass with a 3 square inch circular piston (approximately 2-inch diameter) at the rate of 0.05 inches/minute to (2) the force per unit area required for corresponding penetration of a standard material (FHWA 2010*)*; or an index, from 0 to 100, to measure the supporting capacity of a pavement base, subbase, or subgrade. The lab or field test compares the support capacity of a material with that of a very weak, high plasticity clay soil which has a CBR of 0 to 1 and a high quality, well–graded crushed stone which has a CBR of 100 (modified from pavementinteractive.org)

Cement kiln dust (CKD) – an industrial by-product with constituents including partially calcined and unreacted raw feed, clinker dust, and fuel ash, which is enriched with alkali sulfates, halides and other volatiles.

Chemical stabilization – the mixing of admixtures such as lime, cement, fly ash, and asphalt with subgrade soils to improve strength, improve deformation characteristics, reduce swelling and reduce frost heave.

Design life (design period) – the length of time for which a pavement structure is being designed, including the time from construction until major programmed rehabilitation. (FHWA 2010)

Drainable granular subbase – a subbase constructed of compacted and crushed open graded aggregate, which is highly permeable. (FHWA 2010)

Electro-osmosis – the process which involves movement of pore water in capillaries under the influence of electric potential.

Empirical design – approach is one that is based solely on the results of experiments or experience. Observations are used to establish correlations between the inputs and the outcomes of a process – e.g., pavement design and performance. (FHWA 2010)

Excavation and replacement – method where unsuitable soil, such as soft clay or highly organic soil, under or near a proposed pavement or embankment is removed and replaced by a higher quality material.

Fines content – percentage of the total dry mass of a soil sample that contains grains in the silt and clay range, i.e. particles smaller than US: #200 sieve.

Flexible pavement – in simplest terms, have an asphaltic surface layer, without underlying Portland cement slabs. The asphaltic surface layer may consist of high quality, hot mix asphalt concrete, or it may be some type of lower strength and stiffness asphaltic surface treatment. In either case, flexible pavements rely heavily on the strength and stiffness of the underlying unbound layers to supplement the load carrying capacity of the asphaltic surface layer. A pavement structure that maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability. (FHWA 2010)

Freeze/thaw – the major effect is the weakening that occurs during the spring thaw period. Frost heave during the winter can also cause a severe reduction in pavement serviceability

(increased roughness). The requirements for freeze/thaw conditions are (a) frost-susceptible soil; (b) freezing temperatures; and (c) availability of water. (FHWA 2010)

Full encapsulation - completely surrounding the soil with a geomembranes to prevent moisture from entering or exiting the expansive soil.

Geocell – a three-dimensional comb-like structure, to be filled with soil, aggregate or concrete. (FHWA 2008)

Geocomposite – a geosynthetic material manufactured of two or more materials. (FHWA 2008)

Geogrid (GG) – a geosynthetic formed by a regular network of tensile elements with apertures of sufficient size to interlock with surrounding fill material, used primarily as reinforcement of base and subbase layers and in stabilization of soft subgrade layers. Also used in overlays for asphalt reinforcement. (FHWA 2010)

Geomembrane – an essentially impermeable geosynthetic, typically used to control fluid migration. (FHWA 2008)

Geonet – a geosynthetic consisting of integrally connected parallel sets of ribs overlying similar sets of ribs, for planar drainage of liquids or gases. (FHWA 2008)

Geosynthetic – a planar product manufactured from a polymeric material used with soil, rock, earth, or other geotechnical related material as an integral part of a civil engineering project, structure, or system. (FHWA 2010)

Geosynthetic reinforcement in pavement systems – geosynthetics used in pavement systems between the subgrade and the base to improve and restrain the subgrade and within the base to increase the base stiffness.

Geosynthetic separation in pavement systems – the use of geosynthetics as a physical barrier between base course (especially granular base) and underlying fine-grained soil subgrade to prevent mixing through penetration/intrusion and fine migration by pumping.

Geosynthetics in pavement drainage – geosynthetics materials including geotextiles and geocomposites utilized to provide efficient drainage systems at different layers of the pavement, such as subgrade dewatering, road base drainage, and structure drainage systems.

Geotextile (GT) – a permeable geosynthetic made of textile materials, used as a separator between base, subbase and subgrade layers, used as filters in drainage features, and used in

stabilization of soft subgrade layers. Also used in asphalt overlays as a membrane absorption and/or waterproofing layer. (FHWA 2010)

Lightweight fill – a material with a lower unit weight than conventional earth fill material often utilized to reduce the magnitude of applied loads from embankments.

Lime-treated subgrade – a prepared and mechanically compacted mixture of hydrated lime, water, and soil supporting the pavement system. (FHWA 2010)

Lime-fly ash base (LFB or LFA) – a blend of mineral aggregate, lime, fly ash, and water, combined in proper proportions and producing a dense mass when compacted. (FHWA2010)

Mechanical stabilization – the use of a compacted gravel layer or granular layer in conjunction with non-biodegradable reinforcements to improve roadway support over soft wet subgrades and performance of the base course materials.

Mechanistic–empirical (M–E) design (in pavements) – combines features from both the mechanistic and empirical approaches. The mechanistic component is a mechanics-based determination of pavement responses, such as stresses, strains, and deflections due to loading and environmental influences. These responses are then related to the performance of the pavement via empirical distress models. For example, a linearly elastic mechanics model can be used to compute the tensile strains at the bottom of the asphalt layer due to an applied load; this strain is then related empirically to the accumulation of fatigue cracking distress. In other words, an empirical relationship links the mechanistic response of the pavement to its expected or observed performance. (FHWA 2010)

New construction (in pavements) – the design and construction of a pavement on a previously unpaved alignment. All pavements start as new construction. (FHWA 2010)

Onsite use of recycled pavement materials – the reuse of reclaimed asphalt pavement (RAP), concrete pavement (RCP), and unbound, cement treated and asphalt treated base and subbase materials as base or subbase materials through stabilization, remixing or re-cementing in the construction of new pavements, rehabilitation of existing pavements, and embankments.

Open-graded aggregate base (OGAB) – a crushed mineral aggregate base having a particle size distribution such that when compacted the interstices will provide enhanced drainage properties. Also, known as granular drainable layer, and untreated permeable base (UPB). (FHWA 2010)

Partial encapsulation – the placement of geomembranes horizontally, vertically, or horizontally and vertically combined as barriers to minimize or prevent water from entering or exiting expansive soils.

Pavement performance – measure of accumulated service provided by a pavement (e.g., the adequacy with which it fulfills its purpose). Often referred to as the record of pavement condition or serviceability over time or with accumulated traffic. (FHWA 2010)

Pavement section – the structural components of the pavement system which is a combination of the subbase, base course and surface course placed to support the traffic load and distribute it to the roadbed. (FHWA 2010)

Performance period – the period of time that an initially constructed or rehabilitated pavement structure will last (perform) before reaching its terminal condition when rehabilitation is performed. This is also referred to as the design period. (FHWA 2010)

Permeable base (PB) – a base course constructed of treated or untreated open-graded aggregate. Also, free draining base. (FHWA 2010)

Portland cement concrete (PCC) – a composite material consisting of a Portland or hydraulic cement binding medium and embedded particles or fragments of aggregate. (FHWA 2010)

Prefabricated geocomposite edgedrain (PGED) – an edgedrain consisting of an extruded plastic drainage core covered with a geotextile filter (also known as panel drains or fin drains). (FHWA 2010)

Reclaimed asphalt pavement (RAP) – removed and/or processed asphalt pavement materials containing asphalt and aggregates

Reconstruction (in pavements) – the complete removal of an existing pavement and construction of a new pavement on the same alignment. Except for the demolition of the existing pavement (usually done in stages, i.e., one lane at a time) and traffic control during construction, reconstruction is very similar to new construction in terms of design. (FHWA 2010)

Recycled materials – waste, recycled, reclaimed, and by-product materials are collectively grouped under this general category in construction or reconstruction in the highway environment. (FHWA 2001)

Recycled concrete pavement (RCP) – removed and processed concrete pavement for reuse as aggregates

Recycled concrete materials (RCM) – removed and processed concrete for reuse as aggregates, which included recycled concrete pavements (RCP)

Rehabilitation (in pavements) – the restoration or addition of structural capacity to a pavement. Overlays (either asphalt or Portland cement concrete), crack and seat, and full or partial depth reclamation are examples of rehabilitation construction. (FHWA 2010)

Rigid pavement – in simplest terms, are those with a surface course of Portland cement concrete (PCC). The Portland cement concrete slabs constitute the dominant load carrying component in a rigid pavement system. A pavement structure that distributes loads to the subgrade, having as one course a Portland cement concrete slab of relatively high bending resistance. (FHWA 2010)

Roadbed – the graded portion of a highway between top and side slopes, prepared as a foundation for the pavement structure and shoulder. (FHWA 2010)

Rutting – longitudinal depression or wearing away of the pavement in wheel paths under load. (FHWA 2010)

Soil cement – a mechanically compacted mixture of soil, Portland cement, and water, used as a layer in a pavement system to reinforce and protect the subgrade or subbase. Also, cement treated subgrade (CTS). (FHWA 2010)

Subbase – the layer or layers of specified or selected materials of designed thickness placed on a subgrade to support a base course. Note that the layer directly below the PCC slab is now called a base layer, not a subbase layer. (FHWA 2010)

Subgrade – the top surface of a roadbed upon which the pavement structure and shoulders are constructed with the purpose of providing a platform for construction of the pavement and to support the pavement without undue deflection that would impact the pavement performance, or the natural and/or prepared soil materials beneath the pavement structure that deform under pavement loading or otherwise have an influence on the support of the pavement (a.k.a. roadbed, pavement foundation). (FHWA 2010)

Swelling or expansive soils – swelling refers to the localized volume changes in expansive roadbed soils as they absorb moisture. (FHWA 2010)

Unbound base/subbase – compacted mineral aggregate layer that may be either untreated or treated, but has not been modified sufficiently to provide an increase in stiffness or strength for design. (FHWA 2010)

Unpaved roads (naturally surfaced roads) – simply are not paved, relying on granular layers and the subgrade to carry the load. Seal coats are sometimes applied to improve their resistance to environmental factors. (FHWA 2010)

Vacuum preloading – a technique that induces an increase in effective stress in the saturated soft soils through a reduction in pore pressures and results in consolidation accomplished by creating a vacuum underneath an airtight membrane.

1.6 Primary References

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2.0 MECHANICAL STABILIZATION: THICKER GRANULAR LAYERS AND BLENDING

As indicated in Section 1.0, thick granular layers can be used to provide several benefits, including increased load-bearing capacity, frost protection, and improved drainage. While the composition of this layer takes many forms (e.g., stabilized and unstabilized dense graded gravel, open graded gravel, lightweight materials, recycled granular materials, etc.), the underlying strategy of each is to achieve desired pavement performance through improved foundation characteristics of strength, deformation and/or drainage. This chapter describes the benefits of thick granular layers, typical characteristics, and considerations for the design and construction of granular pavement support.

2.1 Feasibility Considerations

The zone that may be selected for improvement depends upon a number of factors. Among these are the depth of soft soil, strength and modulus of the soft soil, anticipated traffic loads, the importance of the transportation network, constructability, the drainage characteristics of the geometric design and the underlying soil, and depth to water table. When only a thin zone and/or short roadway length needs improvement, removal and replacement will usually be the preferred alternative by most agencies, unless a suitable replacement soil is not economically available. Note that in this context, the use of the qualitative term “thin” is intentional, as the thickness of the zone can be described as thick or thin, based primarily on the project economics of the earthwork requirements and the depth of influence for the vehicle loads.

2.1.1 Applications

Thick granular layers (i.e., in addition to the design base/subbase layer thickness), either over the soft soil or using excavation to a partial or full depth of the soft soil, have been incorporated into pavement design in several ways. They can be referred to as fills or embankments, an improved or prepared subgrade, and select or preferred borrow. Occasionally, a thick granular layer is used as the pavement subbase.

Thick granular layers are also used to avoid or reduce frost problems by providing a protection to the underlying subgrade layers. A common practice in several New England and Northwestern states is to use 3 feet or more of gravel beneath the pavement section. The gravel improves drainage of surface infiltration water and provides a weighting action that reduces and results in more uniform heave. Washington State has successfully used an 18 inches layer of cap rock beneath the pavement section in severe frost regions (Ulmeyer et al. 2002).

On projects where the compacted in situ soil does not quite meet the required strength/ stiffness requirements and appears to be inadequately graded (i.e., too coarse, too fine, or gap-graded), mixing of the in situ soil with a material of different gradation (i.e., blending) may sufficiently improve the support of the pavement section. The process is ideal if suitable materials (e.g., cut/borrow materials) are available on the project. Care needs to be taken to ensure that there are no new performance consequences after blending (e.g., different freeze-thaw behavior after adding fine material). On projects where the blend materials need to be imported, costs will have to be compared with other methods of stabilization. In these instances, chemical stabilization of fine materials (i.e., silts and clays) will usually be cheaper than importing coarse materials, while sourcing and transporting nearby fine material (*fill soil*) to improve the properties of coarse- or gap-graded materials (e.g., single-sized sands) may be less expensive than chemical stabilization.

2.1.2 Advantages and Potential Disadvantages

In areas with large quantities of readily accessible, good quality aggregates, a thick granular layer may be used as an alternative soil stabilization method. The aim of using thick granular layers is to improve the natural soil foundation. In doing this, many agencies are recognizing that the proper way to account for weak, poorly draining soils is through foundation improvement, as opposed to increasing the pavement layer thicknesses. Following are advantages and potential disadvantages in using this stabilization method.

2.1.2.1 Advantages

The following are potential benefits in using thick granular layers:

- Increase the supporting capacity of weak, fine-grained subgrades.
- Provide a minimum bearing capacity for the design and construction of pavements.
- Provide uniform subgrade support over sections with highly variable soil conditions.
- Reduce the seasonal effects of moisture and temperature variations on subgrade support.
- Promote surface runoff through geometric design.
- Improve subsurface drainage and the removal of moisture from beneath the pavement layers, provided free draining granular layers are used.
- Increase the elevation of pavements in areas with high water tables.
- Provide frost protection in freezing climatic zones.
- Reduce subgrade rutting potential of flexible pavements.

- Reduce pumping and erosion beneath PCC pavements.
- Meet elevation requirements of geometric design.

2.1.2.2 Potential Disadvantages

If good quality aggregates are not readily available, thick granular layers and/or blending can be an expensive alternative. Also, the use of poor quality aggregates can lead to inadequate pavement support, insufficient drainage, deterioration over time, and frost susceptibility. If soils are extremely soft, placement of the aggregate may be difficult and a significant amount of gravel may be lost due to penetration of the aggregate into the subgrade and intrusion of the subgrade into the aggregate (e.g., through pumping of fines). This will likely occur when the dumped pile exceeds a height $h > (3c/\gamma)$, where c is the cohesive strength of the subgrade soil and γ is the unit weight of the granular layer.

Over excavation of soft soils may be required, the depth of which may be as thick as the gravel layer, and disposal of the excavated materials must be considered in the cost. Thick granular layers provide a working platform, but do not provide strengthening of the subgrade beneath the gravel layer. In fact, construction of thick granular layers in some cases results in disturbance of the subgrade due to required construction activities for their installation.

2.1.3 Feasibility Evaluations

2.1.3.1 Geotechnical

The sensitivity and strength and depth of soft of the soft subgrade are the key geotechnical parameters. The requirement for an additional gravel layer and the design of its thickness are of course also important geotechnical issues. Geotechnical questions in using the gravel and determining thickness requirements will be related to its purpose (i.e., is the gravel being used to decrease the effects of soft clays or is it being used to mitigate issues with expansive clays or frost heave). For deep soft subgrade soil deposits, the potential for subgrade consolidation is also an important geotechnical consideration. Removing and replacing soft subgrade soils with granular layers should always be considered when subgrade soils contain high organic content (e.g., peat type soils).

To consider blending, candidate subgrade soils should typically have a plasticity index of greater than 12, an Stabilometer R-value (AASHTO T 190 or ASTM D2844) of less than 20, and/or an equivalent Dynamic Cone Penetrometer (DCP) (ASTM D6951) number based on calibration tests. Better soils, (typically GW, GP, and SW soils, and potentially GM, GC, and SP soils), will most likely not need any additional stabilization beyond standard subgrade preparation requirements (i.e., rip and recompact). If the problem is related only to high soil moisture content, check that the drainage design is satisfactory and consider use of blending

to dry out the soil. If the problem is primarily related to soil gradation and not plasticity (typically GP, GC, and SP soils), blending could also be considered. Other conditions where thicker gravel or blending should be considered are where subgrade soils have a high sulfate content, as these soils are likely not suitable for chemical stabilization.

2.1.3.2 Environmental Considerations

Frost susceptibility of the subgrade below the gravel layer and the depth of frost penetration are key parameters to consider in determining the required depth of the granular layer. Blending is usually not a good candidate stabilization method where frost susceptibility is an issue, as the blended material may also likely be frost susceptible. A significant environmental consideration is the material that must be removed, which in some cases may be contaminated or contain organics, which may restrict its disposal. Another environmental concern is dust control.

2.1.3.3 Site Conditions

Clearing and grubbing requirements and required excavation to subgrade depth are important site issues. As indicated under the potential disadvantages discussion, this activity, unless carefully controlled, can disturb the subgrade soils and increase the stabilization requirements to satisfy established design foundation support criterion. Therefore, determining the subgrade type, initial strength/stiffness and its sensitivity are key requirements in evaluating the subgrade conditions. Also, the presence of high water levels, either the groundwater or perched conditions must be determined. Drainage may be critical to construction activities (see Section 8 in this chapter).

2.1.4 Limitations

The primary limitations on use of thick granular layers and/or blending are the availability of readily accessible, good quality aggregates and the ability to dispose over excavated subgrade materials.

Another limitation may be the availability of soft ground construction equipment to over excavate the subgrade and place the gravel, such that this operation can be performed without significantly disturbing the subgrade. In sensitive soils, which can lose significant strength under construction activities, such disturbance can lead to significant over use of gravel. In deep deposits of extremely soft (i.e., CBR < 1) soils, the soil may not support equipment or even the placement of the gravel without bearing failure.

2.1.5 Alternative Solutions (or Technologies)

All of the other stabilization technologies in this chapter should be considered. Complementary technologies include dewatering and the use of geosynthetics with the aggregate, either or both of which will result in a significant reduction in aggregate thickness required to stabilize the subgrade, as well as long-term performance improvements.

2.2 Construction and Materials

2.2.1 Construction

Soft ground construction equipment (e.g., low ground pressure wide track dozers, extended backhoe grade-all excavators, partially loaded dump trucks, etc.) for the over excavation and placement of gravel should be considered to avoid disturbance of the subgrade. In sensitive soils, such disturbance can lead to significant over use of gravel or a section that does not perform as well as anticipated. Any equipment that results in subgrade or granular layer rutting of more than a few inches should not be allowed to operate on the site. Adequate compaction of the stabilization gravel used for the roadbed and the subbase, and base course layers must also be achieved (see Holtz 1990) for information on compaction methods).

2.2.2 Materials

As indicated at the beginning of this section, the granular layer can take on many forms. The two most important characteristics for any of these layers are material properties and thickness. While geometric requirements (e.g., vertical profile) and improved surface runoff can be achieved by embankments constructed of any soil type, the most beneficial effects are produced through utilization of good quality, granular materials. Several methods are used to characterize the strength and stiffness of granular materials, including the California Bearing Ratio (CBR) and resilient modulus testing. In addition, several types of field plate load tests have been used to determine the composite reaction of the embankment and soil combination. In general, materials with CBR values of 20% or greater are used, corresponding to resilient moduli of approximately 17,500 psi. These are typically sand and/or gravel size materials with limited fines, corresponding to AASHTO A-1 and A-2 (ASTM D2487 USCS: GW, GP, SW and SP) soils. Large sized fractions (e.g., up to 4 inches) may also be considered, especially if blended with smaller sized fractions to form a well graded material, or a separation layer is used beneath the coarser material.

Aggregate gradation and particle shape are other important properties. Typically, stabilization layers are dense-graded, with a maximum top-size aggregate that varies depending on the thickness of the layer. Where very thick layers are required, the bottom stabilization lift may contain cobbles or aggregates of 4 to 8 inches in diameter, but no

greater than ½ the lift thickness. Granular layers placed close to the roadbed surface have gradations, including maximum size aggregates, similar to agency required subbase material specifications. Although dense-graded aggregate layers do not provide efficient drainage relative to open-graded materials, a marginal degree of subsurface seepage can be achieved by limiting the fines to non-plastic materials with a fines content of less than 5%. The type of granular material used is normally a function of material availability and cost. Pit-run gravels and crushed stone materials are the most common. The high shear strength of crushed stone is more desirable than rounded, gravelly materials; however, the use of crushed material may not always be economically feasible.

The thicknesses of granular layers vary, depending upon their intended use. Granular layers 6 to 12 inches thick may be used to provide uniformity of support, or act as a construction platform for paving section construction. To increase the composite subgrade design values (i.e., combination of granular layer over natural soil), it is usually necessary to place a minimum of 1.5 to 5 feet of material, depending on the strength of the granular material relative to that of the underlying soil. Likewise, granular fills placed for frost protection may also range from 1.5 to 5 feet. In most cases, thicknesses for greater than 6 feet have diminishing effects in terms of strength, frost protection, and drainage.

2.3 Design Overview

2.3.1 Design Considerations

The use of a thick granular layer presents an interesting situation for design. The placement of a granular layer of substantial thickness over a comparatively weak underlying soil forms, essentially, a non-homogeneous subgrade in the vertical direction. Pavement design requires a single subgrade design value, for example CBR, resilient modulus, or k-value. This is generally determined through laboratory or field tests, when the soil mass in the zone of influence of vehicle loads is of the same type, or exhibits similar properties. In the case of a non-homogeneous subgrade, the composite reaction of the roadbed and soil combination can vary from that of the natural soil to that of the granular layer. Most commonly, the composite reaction is a value somewhere between the two extremes, depending upon the relative difference in moduli between the soil and roadbed material, and the thickness of the granular layer. The actual composite subgrade response is not known until the granular layer is placed in the field, and it may be different once the upper pavement layers are placed (i.e., due to confinement and/or contamination).

To account for non-homogenous subgrades in pavement structural design, it is recommended to characterize the individual material properties by traditional means, such as resilient modulus or CBR testing, and to compare these results to field tests performed over the

constructed embankment layers, as well as the completed pavement section. Analytical models, such as elastic layer programs, can be used to make theoretical predictions of composite subgrade response, and these predictions can then be verified by field testing. Some agencies use in situ plate load tests to verify that a minimum composite subgrade modulus has been achieved. Falling Weight Deflectometer (FWD) and light weight FWD (LFWD), can also be used for testing over the compacted layer and over the constructed pavement surface (e.g., see Vennapusa and White 2009).

It is advisable to use caution when selecting a design subgrade value for a non-homogenous subgrade. Experience has shown that a good-quality granular layer(s) must be of significant thickness, say 3 feet or more, before the composite subgrade reaction begins to resemble that of the granular layer. This means that, for granular layers up to 3 feet in height, the composite reaction modulus can be much less than that of the granular layer itself. If a subgrade design value is selected that is too high, the pavement section will be under-designed. Granular layers less than 1.5 feet thick have minimal impact on the composite subgrade reaction, when loaded under the completed pavement section.

2.3.2 Design Steps

2.3.2.1 Gravel Thickness

Several methods are available for determining gravel thickness for the granular stabilization layer. The stabilization lift can be designed as an unpaved road following the 1993 AASHTO *Guide for Design of Pavement Structures* (AASHTO 1993) for aggregate surfaced roads or designed using haul road design methods, such as those developed by the UAFS (USDA 1977 and USDA 1996) or the United States Army Corps of Engineers (USACE) (Barber et al. 1978).

Rutting is the primary distress for aggregate surfaced roads. Vehicles traveling over aggregate surfaced roads generate significant compressive and shear stresses that can cause failure of the soil. An acceptable rutting depth for aggregate surfaced roads can be estimated considering aggregate thickness and vehicle travel speed. A 2-inch rut depth in a 4-inch-thick aggregate layer probably will result in mixing of the soil subgrade with the aggregate, which will destroy the paving function of the aggregate. Rutting depths greater than 2 to 3 inches in either aggregate or natural surface roads can be expected to significantly reduce vehicle speeds.

The depth of rutting in aggregate surfaced roads will depend upon the soil support characteristics and magnitude and number of repetitions of vehicle loads. The most common measure of rutting susceptibility is the CBR test. Both the CBR test and rutting involve

penetration of the soil surface due to a vertical loading. Although the CBR test does not measure compressive or shear strength values, it has been empirically correlated to rut depth for a range of vehicle load magnitudes and repetitions. The USFS (USDA 1996) uses the following relationship for designing aggregate thickness in aggregate surfaced roads:

$$Rut\ Depth\ (inches) = 5.833\,\frac{(F_r\ R)^{0.2476}}{(\log t)^{0.02}\ C_1^{0.9335}\ C_2^{0.2848}} \qquad \text{[Eq. 9-1]}$$

where,

R = number of equivalent single axle loads (ESALs) at 80 psi tire pressure

t = thickness of top layer (inches)

C_1 = CBR of top layer

C_2 = CBR of subgrade

F_r = reliability factor applied to R (see Table 9-5)

Table 9-5. Reliability Factors for use in Equation 9-1

Reliability Level (%)	Reliability Factor F_r
50	1.00
70	1.44
90	2.32

Equation 9.1 is based upon an algorithm developed by the USACE (Barber et al. 1978). Consult the USFS *Earth and Aggregate Surfacing Design Guide* (USDA 1996) for more details on the design procedure. The allowable ESALs for *R* in Equation [9.1] will vary depending upon the pavement materials and tire pressure. ESAL equivalency factors are defined in terms of pavement damage or reduced serviceability. The USFS Design Guide (USDA 1996) suggests that the ESAL equivalency factor for a 34-kip tandem axle is between 0.09 and 2.15 for tire pressures varying between 25 to 100 psi. According to the AASHTO *Guide for Design of Pavement Structures* (AASHTO 1993), this same axle has equivalency factors of between 1.05 and 1.1 for flexible pavements (*SN* between 1 and 6) and between 1.8 and 2 for rigid pavements (slab thickness *D* between 6 and 14 inches). Rut depth can be managed by limiting tire pressures. Rut depth can decrease by more than 50% for aggregate surfaced roads if the tire pressure for a 34-kip tandem axle is reduced from 100 to 25 psi.

The *South Dakota Gravel Roads Maintenance and Design Manual* (Skorseth and Selim 2000) presents two additional design approaches for aggregate surfaced roads. One approach consists of design catalogs based on traffic categories, soil support classes, and climatic region. The more analytical approach considers ESALs, subgrade resilient modulus, seasonal variations of subgrade stiffness, elastic moduli of the other pavement materials, allowable serviceability loss, allowable rutting depth, and allowable aggregate loss. The loss of thickness due to traffic is unique to aggregate surfacing and must be considered by all thickness design methods for these types of roads. The hardness and durability of the aggregate also require evaluation.

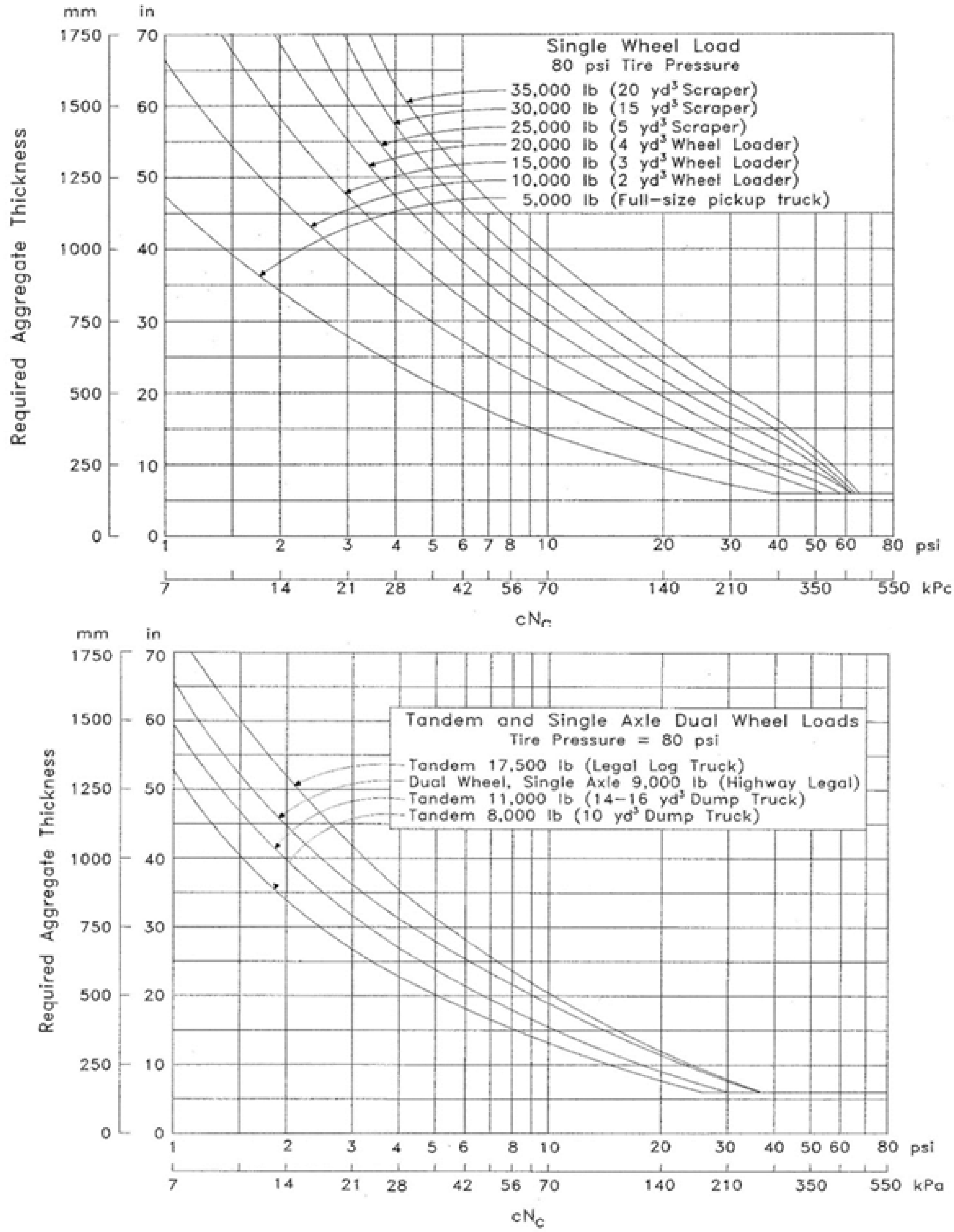
The USFS also has a widely used chart method for direct haul road design, which considers vehicle passes, equivalent axle loads, axle configurations, tire pressures, subgrade strengths and rut depths (USDA 1977). This method is relatively simple and provides well proven results for supporting construction traffic. The following limitations do apply:

- Aggregate layer must be a) high quality fill (e.g., laboratory CBR based on ASTM D1883 $\geq$ 80), b) cohesionless (nonplastic)
- Vehicle passes less than 10,000
- Subgrade undrained shear strength less than about 2000 psf (CBR < 3)

Following are the steps for design based on the USFS approach, which will also be used for design gravel layer thickness when using geosynthetics.

Step 1. Identify properties of the subgrade, including CBR, location of groundwater table, AASHTO and/or USCS classification, and sensitivity.

Step 2. Determine aggregate thickness *t* needed for establishment of a construction platform. The curves shown in Figure 9-1 were developed by FHWA for conventional highway construction traffic based on the USFS's initial curves developed for off road logging trucks. These curves provide aggregate thickness requirements for the expected single or dual wheel load with 80 psi tire pressure and the subgrade bearing capacity.



Adapted from FHWA 2008, after USDA 1977

Figure 9-1. Stabilization aggregate thickness design curves for a single wheel load (top) and dual wheel loads (bottom).

Select the bearing capacity factor N_c based on allowable subgrade ruts, where:

N_c	=	2.8 for a low rut criterion (< 2 inches)
N_c	=	3.0 for moderate rutting (2 to 4 inches)
N_c	=	3.3 for large rutting (> 4 inches)

Alternatively, local policies or charts may be used.

Select the greater value of *t*.

Step 3. Check filtration criteria for the gravel to be used.

$$D_{15}\ of\ aggregate < 5 \times D_{85}\ of\ subgrade \qquad \text{[Eq. 9-2]}$$

and

$$D_{15}\ of\ aggregate > 5 \times D_{15}\ of\ subgrade \qquad \text{[Eq. 9-3]}$$

2.3.2.2 Blending

Soil mixing can alter particle properties to help achieve better stability or drainage. Aggregate gradation is an important factor when engineers design pavement base courses. The following formula is used to express size distribution (Barksdale 1991, Krebs and Walker 1971, and Rollings and Rollings 1996).

$$P = 100\left(\frac{d}{D}\right)^{n} \qquad \text{[Eq. 9-4]}$$

where,

P	=	percent finer than the sieve
D	=	maximum particle size, consistent units with d
n	=	coefficient
d	=	the sieve size, consistent units with D

The maximum density may be achieved when *n* is equal to 0.5. Higher density can help achieve high frictional strength. The grading for maximum density is dependent on aggregate angularity, shape, surface roughness, size, and the method used for construction (Barksdale

1991). In addition, the amount and plasticity characteristics of the fines in an aggregate govern its behavior.

Analytical and graphical design procedures are presented in references (Krebs and Walker 1971, Barksdale 1991, Rollings and Rollings 1996, and Jones et al. 2010). Other methods, such as spreadsheets and computer programs, are also available. It is normal to blend two or more types of soil together in order to meet a certain specification. To successfully use the analytical method, different combinations of soil blending are required to find the desired gradation. The formula below is used to determine the final percentage passing a known sieve size.

$$P = A \times a + B \times b + C \times c + \ldots \qquad \text{[Eq. 9-5]}$$

where,

P	=	percentage of the combined aggregate passing a given sieve size
A,B,C,…	=	percentage of each aggregate (A,B,C,...) that passes a given sieve size
a,b,c,…	=	proportion of each aggregate needed to meet the requirements for material passing the given sieve.

For detailed procedures refer to Krebs and Walker (1971) and Jones et al. (2010). The graphical method also follows the trial and error procedure. An example of mixing two soil materials is described in Krebs and Walker (1971). The advantage of the graphical method is that it is straight forward and easy to use. However, it becomes complicated when more than two soils are blended. Using the analytical procedure can allow for more precise blending and can easily be extended to more than two soils. The precision of blending, however, is usually limited by field conditions.

2.3.3 Primary Design References

2.3.3.1 Thick Gravel

- AASHTO. (1993). *AASHTO Guide for Design of Pavement Structures*. American Association of State Highway and Transportation Officials, Washington, D.C.
- Barber, V.C., Odom, E.C., and Patrich, R.W. (1978). *The Deterioration and Reliability of Pavements*. Technical Report S-78-8, US Army Engineering Waterways Experiment Station, Vicksburg, MS.

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- USDA. (1996). *Earth and Aggregate Surfacing Design Guide for Low Volume Roads*, EM-7170-16, FHWA-FLP-96,001, U.S. Forest Service, U.S. Department of Agriculture.

2.3.3.2 Blending

- Barksdale, R.D. (1991). *The Aggregate Handbook*. National Stone Association, Washington, D.C.
- Jones, D., Tahim, A. Saadeh, S., and Harvey, J. (2010). *Guidelines for the Stabilization of Subgrade Soils in California*, Institute of Transportation Studies, University of California, Davis, CA.
- Krebs, R.D. and Walker, R.D. (1971). *Highway Materials.* McGraw-Hill, New York, NY.
- Rollings, M.P. and Rollings, R.S. Jr. (1996). *Geotechnical Materials in Construction.* McGraw-Hill, New York, NY, pp. 249-312.

2.4 Overview of Construction Specifications and Quality Assurance

2.4.1 Specification Development

Specifications of gradation, compacted CBR of the gravel for specific moisture-density requirements, durability and, of course, thickness are required. As indicated in the design section 2.3, the gravel thickness is based on the CBR of the subgrade, and the design charts are based on a minimum CBR = 80 for the gravel. Gradation of the stabilization gravel is critical to provide separation of the gravel from the subgrade. Maximum rut depth should also be included as a performance measure in the specifications.

If aggregate is available at or near the site, tests should be performed to determine the properties for its suitability and evaluation of thickness requirements. For materials hauled to the site, most state agencies have specifications for aggregate materials and it is strongly recommended that these are used. Quarries will have these materials readily available and the cost will likely be considerably less than that of special gradations.

2.4.2 Summary of Quality Assurance

Primary quality control considerations will be moisture, density or stiffness, and thickness. Modern GPS methods can be used to control thickness, however, point checks should be made to determine the reliability of GPS measurements. The DCP test can also be used to determine the strength and effective depth of the in-place aggregate. Proof rolling should be performed to evaluate deformation responses and potential for rutting.

2.4.3 Summary of Instrumentation Monitoring and Construction Control

Monitoring will typically consist of measuring rut depth (based on design requirements). Proofrolling could be performed with instruments to monitor deformation and rutting for a more accurate assessment of the improved condition. Ideally, intelligent compaction methods can be used to provide a color map of the areas that require improvement and then be used to confirm the uniformity of the improved layer as well as provide an idea of the composite stiffness achieved for design (see *GeoTechTools* for information on intelligent compaction). If the aggregate will be incorporated into the pavement design, then FWD or LFWD tests can be used to determine the equivalent subgrade stiffness value, provided gradation is adequate to prevent long-term degradation.

2.5 Cost Data

2.5.1 Cost Components

The cost components include the granular material (as measured by the ton) and the excavation and disposal requirements (also measured by the ton). The cost of granular material varies widely throughout the country, typically ranging from $7 to $20/ton and will be influenced by specifications and hauling distance and conditions. The equipment used to clear the subgrade, place and compact the aggregate is common to highway construction projects; therefore, additional mobilization costs are negligible. However, extras may be incurred if light construction equipment such as wide track, low ground pressure dozers, grade-all excavators, partially loaded dump trucks, scrappers, etc. is required due to potential effects on construction rates.

For more cost information and a conceptual cost estimating tool see *GeoTechTools*.

3.0 MECHANICAL STABILIZATION: GEOSYNTHETICS

3.1 Feasibility Considerations

Geosynthetics can be used most anyplace where more gravel would be used and can allow gravel to be placed on a much weaker soil than would be possible for the gravel alone. Geosynthetics are placed on weak, often saturated subgrades in combination with a granular layer to stabilize the subgrade and to provide a working platform for construction of unpaved roads, flexible pavements, and rigid pavements. The granular layer may be the base or subbase component of the pavement system or additional gravel may be required to support initial construction and not considered as part of the pavement section. Soft, weak subgrade soils provide very little lateral restraint. When roadway aggregate moves or shoves laterally, vertical deformation occurs and ruts develop on the aggregate surface. A geogrid with good interlocking capabilities or a geotextile with good frictional capabilities can provide *reinforcement* in the form of tensile resistance to lateral aggregate movement. The reinforcement provides lateral restraint and initiates increased horizontal stress on the aggregate, thus reducing the mobilization of the soil and reducing plastic deformations (i.e., reduced rutting). The geosynthetic also increases the system bearing capacity (i.e., stabilization aggregate plus geosynthetic) by forcing the potential bearing surface under the wheel load to develop along an alternate, longer path and thus mobilizing greater shear resistance against rutting.

Other geosynthetic functions play an equal, if not more important, role in this application. The geosynthetic must also provide *separation* to prevent intermixing of the granular material and the subgrade. Only a small amount of fines penetrating into the granular layer will negatively affect its structural characteristics in terms of reduced shear strength and modulus, lowered permeability and increased frost susceptibility. Separation is provided directly by geotextiles or by composite geosynthetics. Geotextiles perform this function by preventing penetration of the aggregate into the subgrade (localized bearing failures) and intrusion of subgrade soils up into the base course aggregate. Geogrids can also prevent aggregate penetration into the subgrade, depending on the ability of the geogrid to confine and prevent lateral displacement of the base/subbase. While the geogrid does not prevent intrusion of subgrade soils up into the base/subbase course, the confinement provided by the geogrid helps to maintain the filtration characteristics of a granular layer designed to provide this function. If the granular material does not meet filtration/separation requirements (e.g., open graded gravel), then a geotextile must be used in combination with the geogrid (i.e., placed separately directly beneath the geogrid or as a geocomposite).

The subgrade soils in this application are fine grained soils with high water content. Therefore, the geosynthetic must also provide *filtration*, i.e., adequate water flow capacity

without clogging, to allow excess pore water pressure to dissipate into the aggregate base course such that destabilizing pore pressure in the subgrade generated from wheel loads can rapidly dissipate. A geotextile used as the primary stabilization geosynthetic will directly provide filtration. In the case of geogrids, either a geotextile or a graded granular separation layer must provide the filtration. This is an important consideration for both construction and long-term support, as pore water pressure dissipation will also allow for strength gains in the subgrade over time. Water can also flow in the plane of a needle-punched nonwoven geotextile. Therefore, in cases of poor-quality aggregate, pore water pressure could be dissipated through the plane of the stabilization geotextile or separation geotextile used with a geogrid if a needle-punched nonwoven geotextile is used. Alternatively a geocomposite that provides in-plane drainage could be considered for the stabilization layer.

It is the reinforcement, separation, filtration and drainage functions that combine to provide the mechanical stabilization for weak subgrade soils (FHWA 2008). AASHTO M 288 (AASHTO 2014a) has identified this geosynthetic mechanical stabilization to be most applicable in subgrade soils where the undrained shear strength is less than about 2000 psf (CBR about 3). Some rutting will probably occur in such soils, especially after a few hundred passes (Webster 1993), but stabilization lifts with only a few passes can be designed with little or no rutting for optimum pavement support.

For higher strength soils (i.e., a CBR $\geq$ 3) that are seasonally weak (e.g., from spring thaw) or for high fines content soils which are susceptible to pumping, a geotextile separator may be of benefit in preventing migration of fines at a much higher subgrade undrained shear strength. This is especially the case for permeable base applications. For soils with a CBR $\geq$ 3, the geotextile application is identified as separation in the AASHTO M 288 specification, recognizing this greater range of geotextile applicability.

3.1.1 Applications

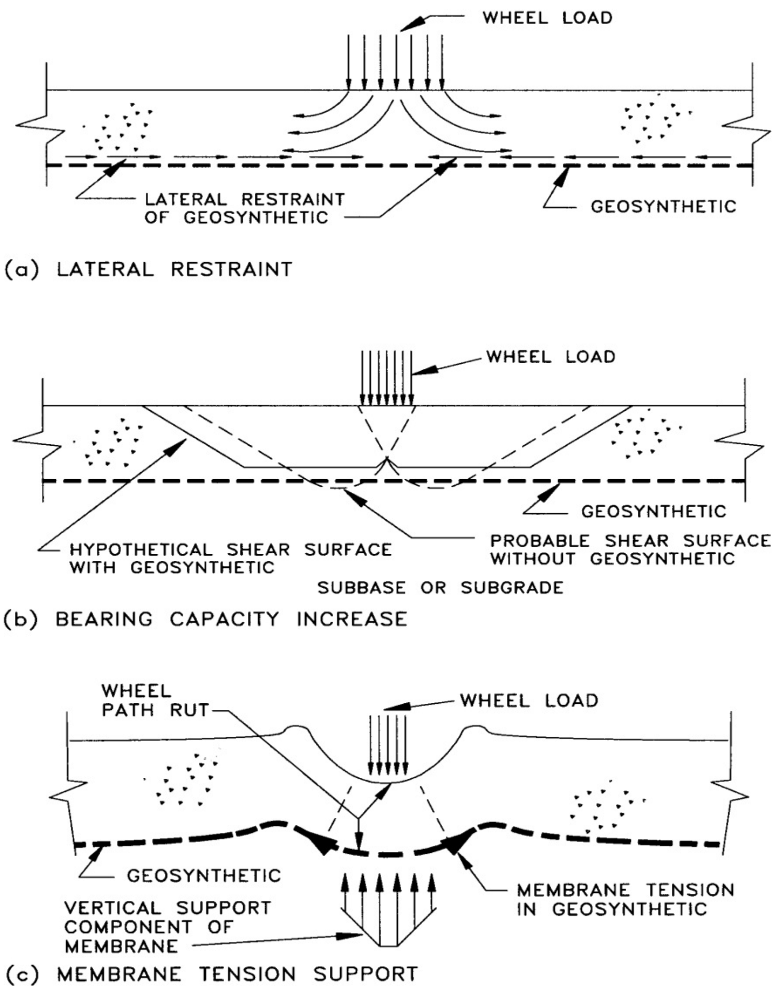
Geosynthetics can be used to enhance the performance of aggregate in most any application where gravel is used. Where soft subgrade soils are present (CBR < 3), geosynthetics can be used to reduce stabilization gravel and enhance the improvement to the subgrade. Where marginal subgrades are present (e.g., CBR $\approx$3 to 4), geosynthetics can be used directly between the base or subbase layer and the subgrade as a stabilization layer to provide improved construction and long-term support. They can be used as a separation layer for even higher strength subgrade containing fines (up to CBR = 8) to maintain the integrity of the aggregate over the life of the pavement system. They are especially effective when free draining, more open graded gravel is used.

3.1.1.1 Separation

Geotextiles used as separators over high fines subgrades prevent the migration of fines into stabilization layers as well as base or subbase when placed directly over the subgrade. This is an especially important application where free draining (i.e., open graded) aggregate is used. In either case, the separation layer both maintains the effective thickness of the aggregate layer and, in the second case, the drainage characteristics of the aggregate over the life of the pavement. In essence, the geosynthetic improves the reliability of the pavement system performance. Thus, the geosynthetic will ultimately increase the life of the roadway.

3.1.1.2 Stabilization (A Combination of Reinforcement, Separation and Filtration)

Geosynthetics are primarily used in stabilization applications to facilitate construction. The geosynthetic provides lateral restraint of the aggregate (see Figure 9-2, top), reducing rutting and/or allowing a reduction in thickness of stabilization/construction aggregate. Even if the finished roadway can be supported by the subgrade, it may be virtually impossible to begin construction of the embankment or roadway without some form of stabilization. The geosynthetic also provides reinforcement to prevent localized bearing failure of aggregate (see Figure 9-2, middle). If significant rutting develops (greater than about 4 inches), the geosynthetic is tensioned, providing additional support by membrane support as shown in Figure 9-2 (bottom). This mechanically stabilized layer also enables contractors to meet compaction specifications for the first two or three aggregate lifts.



FHWA 2008, after FHWA 1981

Figure 9-2. Possible reinforcement functions provided by geosynthetics in roadways: lateral restraint (top), bearing capacity increase (middle), and membrane tension support (bottom).

While the stabilization application is primarily used for initial construction, geosynthetics also provide long-term benefits and improve the performance of the roadway over its design life. As with separation applications, the geosynthetic continues to perform by maintaining the roadway design section and the base course material integrity by preventing the aggregate from penetrating into the subgrade. In addition, the separation function provided by geotextiles, geogrid/geotextile composites or geogrids with appropriately designed filter aggregate prevent the migration of fines into base/subbase materials maintaining the support and drainage characteristics of the base over the life of a pavement system. Again, as with separation, geosynthetics used for stabilization improve the long-term performance of the pavement by maintaining the integrity of the pavement system. In addition, stabilization

geosynthetics provide improved foundation support, especially during overload and/or seasonally weak subgrade conditions.

3.1.2 Advantages and Potential Disadvantages

3.1.2.1 Advantages

Geosynthetics used as separators provide cost and performance benefits when used in roadways with firm, fairly competent subgrades (CBR ranging from 3 to 8), but containing a high quantity of fines, which are often sensitive to seasonal environmental conditions, including the following:

1. Preventing aggregate placed directly on the subgrade (i.e., base, subbase or stabilization aggregate) from penetrating into the subgrade.
2. Preventing subgrade fines from pumping or otherwise migrating up into the aggregate layer.
3. Preventing contamination may allow more open-graded, free draining aggregates to be considered in the design.

In addition to the benefits for separation, geosynthetics used for stabilization of soft to marginal subgrades, CBR < 3 to 4, may provide several additional cost and performance benefits, including the following:

1. Reducing the thickness of aggregate required to stabilize soft subgrades.
2. Reducing the depth of excavation required for the removal of soft, unsuitable subgrade materials.
3. Reducing the intensity of stress on the subgrade.
4. Reducing disturbance of soft or otherwise sensitive subgrade during construction.
5. Producing an increase in soft subgrade strength over time.
6. Providing more uniform support by reducing the differential settlement of roadways constructed over variable subgrade (i.e., soft to firm) conditions and in transition areas from cut to fill, which helps maintain pavement integrity and uniformity. (NOTE: Consolidation settlements are not reduced by the use of geosynthetic reinforcement.)

Both separation and stabilization applications have the potential to reduce maintenance requirements, extend the life of the pavement system, and maintain the long-term integrity of base/subbase layer(s) for pavement surface rehabilitation projects.

3.1.2.2 Potential Disadvantages

There are a few challenges that should be considered with geosynthetic use including:

1. Requires long-term cost benefit assessment for some applications. In many cases the cost of the geosynthetic is covered by the savings in aggregate, however, in some applications the use of a geosynthetic will increase the construction cost.
2. There are a number of geosynthetics, some of which will work very well under some sets of conditions, and not so well under others. Laboratory performance tests and/or test sections may be required to select the most appropriate material(s) for a project.
3. Some special procedures are required with respect to storage, handling, and placement of geosynthetics as well as compaction above the geosynthetic. Particular care must be taken during construction to prevent damage to the geosynthetic.

3.1.3 Feasibility Evaluations

3.1.3.1 Geotechnical

The geotechnical considerations identified in the use of thicker gravel layers for stabilization also apply to geosynthetics. In addition, based on 40 years of experience in using these materials, the following subgrade conditions are considered optimum for using geosynthetics in roadway construction:

- Poor soils: USCS: SC, CL, CH, ML, MH, OL, OH, and PT; AASHTO: A-5, A-6, A-7-5, and A-7-6
- Low undrained shear strength:
    - $\tau_f = c_u < 2000$ psf, CBR < 3 (Note: Soaked Saturated CBR as determined with ASTM D4429)
    - R-value (California) ≈ < 20, M_R ≈ < 4500 psi
- High water table
- High sensitivity

Significant fines migration has been observed with a subgrade CBR as high as 8 (Al-Qadi et al. 1998). On firm subgrades, a geotextile placed between the base/subbase layer(s) and subgrade containing high fines content functions as a separator and filter.

3.1.3.2 Environmental Considerations

Environmental considerations are similar to using gravel alone as the stabilization layer. Frost susceptibility of the subgrade and the depth of frost penetration must be considered in determining the required depth of the granular layer. As a result, the thickness requirement for frost heave may be greater than stabilization requirements for improved subgrade strength. Also, any material required to be excavated in order to obtain an adequate stabilization aggregate depth must be evaluated for potential contamination that would restrict disposal of these materials. Dust control is still a requirement, if the aggregate contains fines. Disposal of excavated material is also an environmental site consideration.

3.1.3.3 Site Conditions

Site condition requirements are similar to thicker gravel stabilization layers, including: clearing and grubbing requirements; location of the water table and subgrade drainage requirements; and, definition of subgrade type, strength, stiffness and sensitivity. One of the advantages in using a geosynthetic is that disturbance of the subgrade is better controlled, as discussed in the construction section. The potential of on-site disposal of excavated material should be considered as part of the site investigation. For deep soft soil deposits, the potential for consolidation is also an important factor along with the impact of settlement on adjacent facilities (e.g., existing roads in widening projects).

3.1.4 Limitations

One problem is the absence of a standardized method to evaluate the differences between geosynthetic products (e.g., geogrids or geotextiles). Mechanistic-Empirical design has the potential to solve that dilemma, but until then test or demonstration sections are recommended (e.g., see AASHTO 2014b). Both types of geosynthetics work well in some subsurface conditions, but in some conditions, one will tend to outperform the other. Geocomposites (e.g. geotextile/geogrid – see section 3.2.2.3 for others) tend to work the best in most stabilization applications, albeit at a greater material cost. For any geosynthetic product, care must be taken during installation to prevent damage to the geosynthetic. Geosynthetics have been used on thousands of projects for over 40 years and the long-term data is very promising and, in some cases, shows substantial long-term benefit. However, additional life cycle cost benefit data is needed to support and expand the limited data sets.

3.2 Construction and Materials

3.2.1 Construction

Guidance for construction of geosynthetic stabilization layers is well covered in AASHTO (2014a) and FHWA (2008). The essential important steps are as follows:

Step 1. Ground surface preparation where the geosynthetic is to be placed:

- Perform undercut excavation as required.
- During surface preparation, minimize subgrade disturbance. Only equipment that results in little (e.g., less than 2 inches) or no ruts should be allowed in these areas (e.g., lightly loaded trucks and wide track dozers).

Step 2. Observe the geosynthetic being rolled out over the site. The geosynthetic should not be dragged across the subgrade.

- Check to make sure that the geosynthetic is placed flat against the subgrade soil and pulled taut to remove wrinkles, folds or creases in the geosynthetic. Staples or pins can be used to hold it in place until covered.
- Check the geosynthetic for any damage or defects (e.g., holes, rips or tears). Repair (see item 4) or replace.
- Adjacent geosynthetics are to be overlapped (at least 12 inches or greater per specification requirements) unless sewn seams are used.
    - For overlap seams, check to see that the overlap (the upper geosynthetic) is in the direction the material will be placed to prevent shoving and lifting the edge of the top geosynthetic and that overlap is maintained during fill placement.
    - For sewn seams, check that stitches are up (visible).
- Equipment is not allowed to operate on geosynthetics until covered with specified minimum lift thickness of aggregate fill.

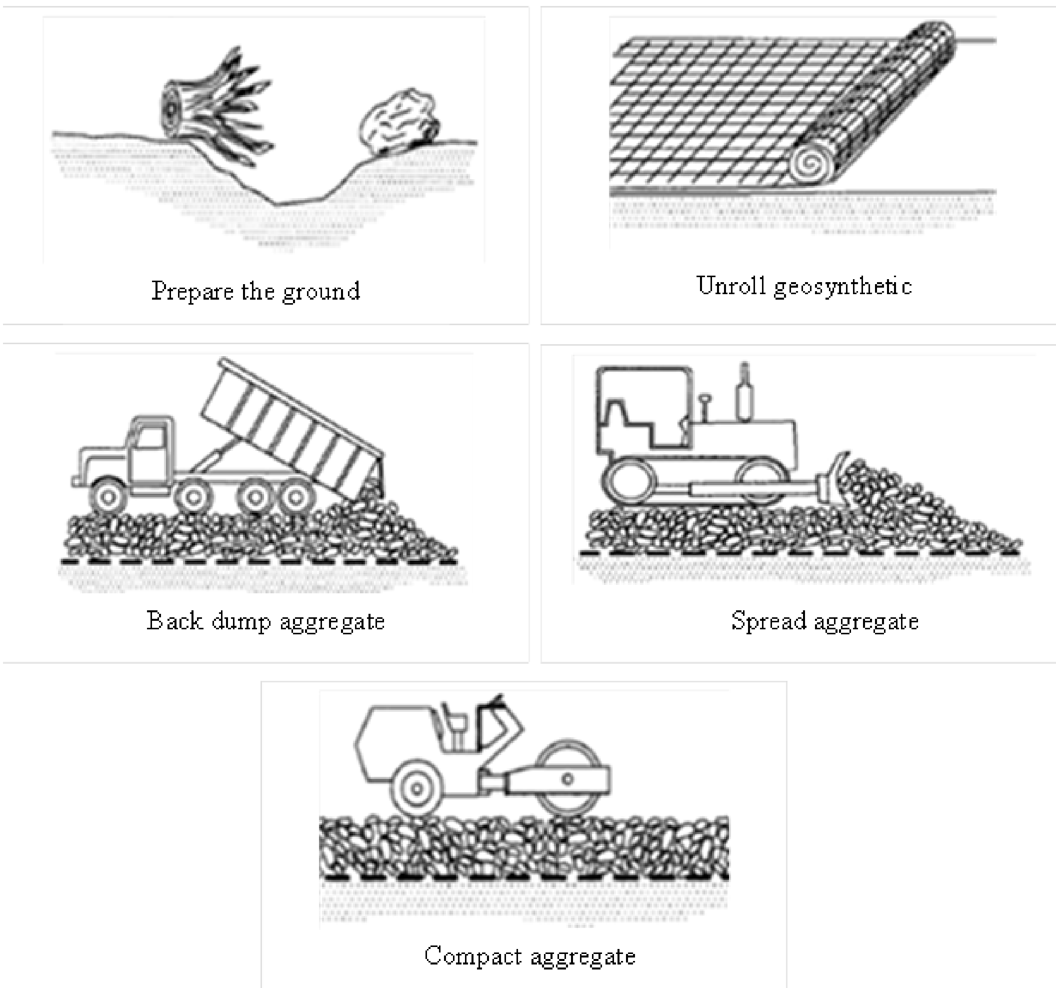
Step 3. Placement and compaction of fill (check that the fill meets specification requirements).

- Fill is to be end dumped onto the previously placed aggregate, spread and graded to the required lift thickness.
- If excessive rutting occurs (more than 3 inches), increase lift thickness or decrease vehicle weight.

- On the first lift for very soft ground, do not use vibratory compaction equipment.
- Fill in any ruts and never blade them down as this decreases lift thickness (see Figure 9-5).

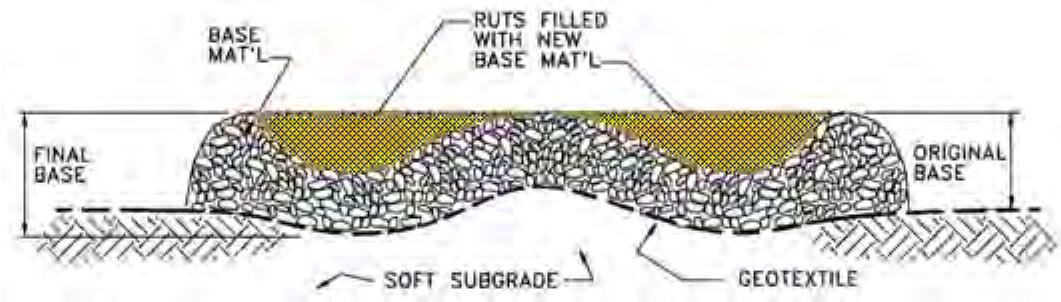
Step 4. In one small section remove fill material from above the geosynthetic and check for damage. If damaged, the contractor should modify placement techniques. Repair any damaged area by either replacing the geosynthetic, or using a patch. The patch should extend at least 18 inches outside of the damaged area.

These four steps are illustrated in Figures 9-3 and 9-4.



FHWA 2008

Figure 9-3. Installation of geosynthetics in soil stabilization: prepare the ground (upper left), unroll geosynthetic (upper right), back dump aggregate (middle left), spread aggregate (middle left), and compact aggregate (bottom).



After FHWA 2008

Figure 9-4. Fill in ruts – do not blade down.

3.2.2 Materials

As with any geosynthetic application, the material properties required for design are based on: 1) the properties required to perform the primary and secondary function(s) for the specific application over the life of the system, and 2) the properties required to survive installation. The separation and filtration functions are related to opening characteristics and are determined based on the gradation of the adjacent layers (i.e., subgrade, base and/or subbase layers). Some strength is, of course, required. If the roadway system is designed correctly, then the stress at the top of the subgrade due to the weight of the aggregate and the traffic load should be less than the bearing capacity of the soil times a safety factor, which is generally a relatively low value compared to the strength of most geosynthetics. However, the stresses applied to the subgrade and the geosynthetic during construction may be much greater than those applied in-service. Therefore, the strength of the geotextile or geogrid in roadway applications is usually governed by the anticipated construction stresses and the required level of performance. This is the concept of geosynthetic survivability, that the geosynthetic must survive the construction operations if it is to perform its intended function. In fact, for subgrade stabilization, the geosynthetic survivability tends to control the strength requirements.

FHWA (2008) and AASHTO (2014a) provide tables specifically for stabilization applications that relate geosynthetic index properties defined by the ASTM standards (i.e., grab strength, CBR puncture resistance, and tear resistance for geotextiles; and, wide width strength and strength for geogrids) to survivability of geosynthetics. These charts are included in the design section (Section 3.3.2). The geosynthetics are classified as High (Type 1), Moderate (Type 2) and Low (Type 3) survivability geosynthetics and the types are matched to specific installation conditions. Opening characteristics for geogrids based on the relation to the granular layer particle size and for geotextiles based on separation and filtration requirements are also included in the tables, plus permittivity requirements are specified for geotextiles.

Other properties, such as stiffness, aperture size and interlock effect, may be required for the specific design methods. Almost no correlations have been developed between properties and field performance of geosynthetics in subgrade stabilization applications. In order to develop such correlations, Berg et al. (2000) recommended that the following properties of interest be provided with any future full scale studies or long-term pavement studies: 2% and 5% secant moduli, coefficient of pullout interaction, coefficient of direct shear, aperture size, and percent open area.

3.2.2.1 Geotextiles

Most important for this application is the strength of the geotextile, which must be strong enough to withstand truck traffic over the aggregate, even when some rutting occurs. If the soil is wet, the geotextile must also let water drain through it so that a waterbed effect is not created, but not allow the soft soil to squeeze through its openings (pores).

Strength of the geotextile is required primarily for construction survivability and is measured based on the grab strength, puncture strength and tear strength of the material as provided in the design section (Section 3.3.2). Elongation at break is related to the stiffness of the material with elongation less than 50% at break typical for woven geotextiles and greater than 50% typical for nonwoven geotextiles. The more a material with a given strength will elongate, the better it will give under load and survive construction. Hence, strength requirements will be greater for woven geotextiles than for nonwoven geotextiles.

The openings in a geotextile are characterized by its apparent opening size, or AOS, which is essentially the largest opening(s) in the geotextile. AOS is used for design to assure that the soil particles larger than the AOS will not go through the geotextile. The flow capacity required for the stabilization application is measured by the permittivity of the geosynthetic and is equivalent to the gallons of liquid that can flow through a square yard of the material.

Ultraviolet light stability is also required to minimize the loss of strength during sunlight exposure. The geotextile should be covered as soon as possible after placement to limit this exposure.

Long-term durability is related to the polymer type (either polypropylene or polyester) and the type of additives used in the polymer. For most roadway applications, the durability life of the geosynthetic should be well in excess of 1000 yrs. One concern would be for polyester geotextiles in high pH (pH > 9) environments (i.e., geotextiles used in conjunction with lime or cement stabilization). For these cases, polypropylene geotextiles should be used.

3.2.2.2 Geogrids

Geogrids are characterized by the manufacturing process, the polymer type and opening characteristics. Geogrids may be extruded, woven or welded. For woven geogrids, a coating is usually applied to maintain stability and protect the fiber strands from construction damage.

Strength of the geogrid is primarily for construction survivability and is measured based on the wide width strength and the junction strength, as provided in the design section (Section 3.3.2). The apertures (i.e., openings) in a geogrid are directly measured and are used in design to assure compatibility with the gravel layer above the geogrid in terms of interlock (i.e., lateral restraint). If the apertures are too small, the gravel could slide over the geogrid, and if they are too large, the interlock and confinement are diminished.

Ultraviolet light stability again is required to minimize the loss of strength during sunlight exposure. Due to less surface area exposure (i.e., the surface area of a single geogrid rib versus many filaments in a geotextile of similar strength), geogrids are less susceptible to UV degradation than geotextiles made of the same polymer type. Even so, the geogrid should also be covered as soon as possible after placement to limit this exposure.

As with geotextiles, long-term durability of geogrids is related to the polymer type (either polyethylene, polypropylene or polyester) and the type of additives used in the polymer. The same concern exists for high pH (pH > 9) environments and polypropylene or polyethylene geogrids should be used in these applications.

3.2.2.3 Geocomposites

Geocomposites used in stabilization applications include woven/nonwoven geotextile composites, geotextile/geogrid composites, nonwoven geotextiles/high strength/modulus inline filaments, and geotextile/geonet composites. In addition to the survivability, opening and durability requirements for either the geotextile or geogrid components mentioned above, other properties are related to bonding of the components (which must be sufficient to maintain their integrity during installation and service) and, in the case of drainage composites, the required compressibility and hydraulic considerations as covered in Section 8. Sliding of one geosynthetic over another has been found to reduce the performance of the composite material in stabilization applications (Christopher and Schwartz 2010).

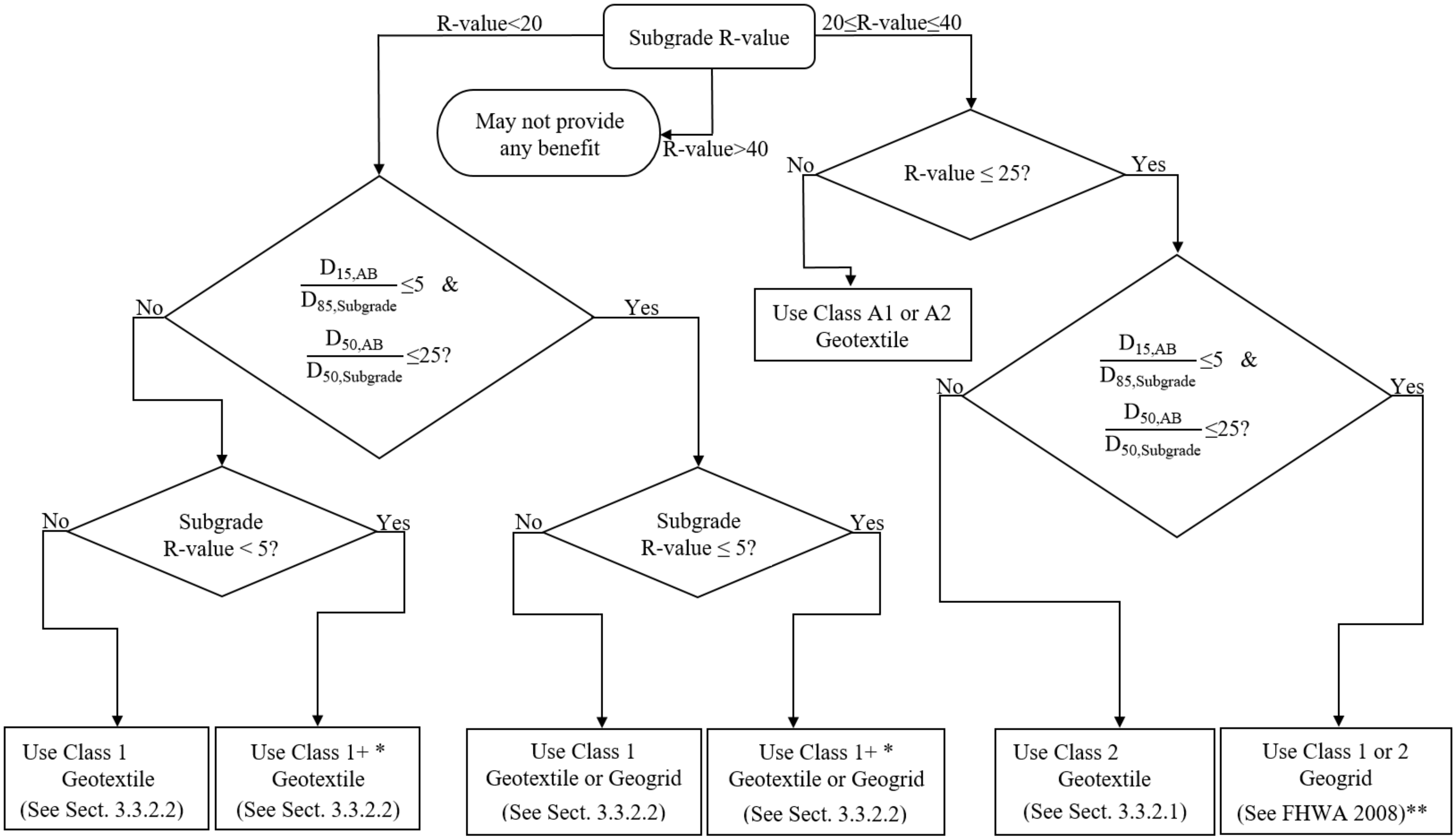
3.3 Design Overview

3.3.1 Design Considerations

As a separator the woven or nonwoven geotextile must prevent the intermixing of the permeable base and the adjacent subgrade or subbase layer. Also, the geotextile layers will have to satisfy filtration criteria. Thus, the geotextile filtration characteristics must be checked for compatibility with the gradation and permeability of the subgrade. In order to provide support for construction traffic, the geotextile must also satisfy survivability and endurance criteria. Design requirements are outlined in FHWA (2008) and below in Section 3.3.2.2.

In stabilization applications, the geosynthetic must also provide separation. However, design requirements for sufficient characteristics to provide adequate filtration considering wet, often saturated subgrade conditions and reinforcement, which are related to stronger survivability and endurance criteria, are also required. The design of the geosynthetic for stabilization is accomplished using the design-by-function approach in conjunction with AASHTO M 288 (AASHTO 2014a), in the steps from FHWA (2008) outlined below in Section 3.3.2.2. A key feature of this method is the assumption that the structural pavement design is not modified at all in the procedure; however, improvements can be made to the subgrade support value, provided certain design measures are followed.

To assist in selecting the appropriate geosynthetic for subgrade stabilization, Caltrans (2014), developed the flow chart in Figure 9-5.



Class 1 and Class 2 are AASHTO (2014a) Geotextile classes (Tables 9-6 and 9-7 later in this chapter) and FHWA (2008) Geogrid classes (Table 9-8 later in this chapter) and Class 1+ are stronger geotextiles than AASHTO Class 1 recommended by Caltrans.

After Caltrans 2014

Figure 9-5. Selecting an appropriate subgrade stabilization geosynthetic.

The selection is based on the strength and gradation of the subgrade as previously reviewed in Section 3.1.3 and the gradation compatibility of the adjacent aggregate base/subbase course layer to prevent migration of subgrade fines. Any additional aggregate requirements for stabilization and specific geosynthetic requirements can then be determined following the design steps in Section 3.3.2.

The pavement design proceeds exactly according to standard procedures, as if the geosynthetic was not present. The geosynthetic instead replaces additional unbound material that might be placed to support construction operations, and replaces no part of the pavement section itself. However, this unbound layer will provide some additional support. If the soil has a CBR of less than 3, and the aggregate thickness is determined based on a low rutting criterion in the following steps, the support for the composite system is theoretically equivalent to a CBR = 3 (resilient modulus of 4500 psi). As with thick aggregate fill used for stabilization, the support value should be confirmed through field testing using, for example, a plate load test or FWD test to verify that a minimum composite subgrade modulus has been achieved. Note that the FHWA procedure is controlled by soil CBR, as measured using ASTM C4429.

3.3.2 Design Steps

3.3.2.1 Separation

The requirements for proper performance of geotextiles used in separation applications can be appropriately selected by using the following design steps from FHWA (2008):

Step 1. Assess the need for a geotextile separator. Determine if subgrade conditions warrant the use of a geotextile separator, which is essentially subgrades with high fines content (i.e., Cl, CH, ML, MH, CM, GM, OL and OH) and CBR ≈ 3 to 8 (M_R ≈ 4500 psi to 12,000 psi, Su ≈ 2000 psf to 5000 psf, or R ≈ 18 to 37).

Step 2. Determine the maximum opening and minimum permittivity requirements based on the gradation of the subgrade. AASHTO M 288 specifications (2014a) require the geotextile to have the drainage and filtration properties shown in Table 9-6.

Table 9-6. Geotextile Property Requirements for Separation Applications (CBR ≥ 3)

Requirement	Property	ASTM Test Method	Units	Requirement for Geotextile Class 2[1] and Elongation < 50%[2]	Requirement for Geotextile Class 2[1] and Elongation > 50%[2]
Survivability	Grab Strength	D4632	lb (N)	250 (1100)	157 (700)
Survivability	Sewn Seam Strength[3]	D4632	lb (N)	220 (990)	140 (630)
Survivability	Tear Strength	D4533	lb (N)	90 (400)	56 (250)
Survivability	Puncture Strength	D6241	lb (N)	495 (2200)	309 (1375)
Survivability	Ultraviolet Stability	D4355	%	50% retained strength after 500 hours of exposure	50% retained strength after 500 hours of exposure
Drainage and Filtration[4]	Apparent Opening Size	D4751	mm	< 0.6 for < 50% passing No. 200 sieve; < 0.3 for > 50% passing No. 200 sieve	< 0.6 for < 50% passing No. 200 sieve; < 0.3 for > 50% passing No. 200 sieve
Drainage and Filtration[4]	Permittivity	D4491	sec^{-1}	> 0.02 and > Permittivity of soil	> 0.02 and > Permittivity of soil

Source: FHWA 2008 (after AASHTO 2014a, but pre-2008 version of these AASHTO specifications)

[1] Default geotextile selection. The engineer may specify a Class 3 geotextile for moderate survivability conditions, see AASHTO M 288.
[2] As measured in accordance with ASTM D4632.
[3] When seams are required. Values apply to both field and manufactured seams.
[4] Also, the geotextile permeability should be greater than the soil permeability.

The following should be noted for Table 9-6:

- Acceptance of geotextile material shall be based on ASTM D4759.
- Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D4354.
- All values are minimum values; use the value in the weaker principal direction. All numerical values represent minimum average roll values (i.e., test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D4354.

The criteria shown in Table 9-6 should be evaluated with respect to filtration design requirements to confirm that the apparent open size (AOS), permeability (k), and permittivity (ψ) requirements are adequate for the specific subgrade soils, especially when geotextile is used as separation for open graded or otherwise free draining base layers as follows (after FHWA 2008):

$$AOS \leq D_{85\ Subgrade}\ (Wovens) \qquad \text{[Eq. 9-6]}$$

$$AOS \leq 1.8 \times D_{85\ Subgrade}\ (Nonwovens) \qquad \text{[Eq. 9-7]}$$

$$k_{Geotextile} > k_{Soil}\ and\ \psi \geq 0.1\,\sec^{-1} \qquad \text{[Eq. 9-8]}$$

When using equation 9-7 for noncohesive silts and other highly pumping susceptible soils, a filter bridge may not develop, especially under dynamic, pulsating flow. A conservative (smaller) AOS ≤ $D_{85subgrade}$ is advised, and laboratory filtration tests are recommended.

Step 3. Determine geotextile survivability requirements. AASHTO M 288 (2014a) provides the criteria for geotextile strength required to survive construction of roads, as shown in Table 9-6. Use Class 2 where a moderate level of survivability is required (i.e., for subgrade CBR > 3), where at least 6 inches of base/subbase and normal weight construction equipment are anticipated, and where filters are used in edgedrains. Class 1 (see Table 9-7) geotextiles are recommended for CBR < 3 and when heavy construction equipment is anticipated. For stabilization and separation geotextile layers, a minimum of 6 inches of base/subbase should be maintained between the wheel and geotextile at all times.

Table 9-7. Geotextile Property Requirements for Stabilization Applications (CBR< 3)

Requirement	Property	ASTM Test Method	Units	Requirement for Geotextile Class 1[1] and Elongation < 50%[2]	Requirement for Geotextile Class 1[1] and Elongation > 50%[2]
Survivability	Grab Strength	D4632	lb (N)	315 (1400)	200 (900)
Survivability	Sewn Seam Strength[3]	D4632	lb (N)	280 (1260)	180 (810)
Survivability	Tear Strength	D4533	lb (N)	110 (500)	80 (350)
Survivability	Puncture Strength	D6241	lb (N)	620 (2750)	433 (1925)
Survivability	Ultraviolet Stability	D4355	%	50% retained strength after 500 hours of exposure	50% retained strength after 500 hours of exposure
Drainage and Filtration[4]	Apparent Opening Size	D4751	mm	< 0.43 for < 50% passing No. 200 sieve < 0.3 for > 50% passing No. 200 sieve	< 0.43 for < 50% passing No. 200 sieve < 0.3 for > 50% passing No. 200 sieve
Drainage and Filtration[4]	Permittivity	D4491	sec^{-1}	0.5 for < 15% passing No. 200 sieve 0.2 for 15 to 50% passing No. 200 sieve 0.1 for > 50% passing No. 200 sieve	0.5 for < 15% passing No. 200 sieve 0.2 for 15 to 50% passing No. 200 sieve 0.1 for > 50% passing No. 200 sieve

Source: AASHTO 2014a

[1] Default geotextile selection. The engineer may specify a Class 2 geotextile for moderate survivability conditions (see Table 9-9).
[2] As measured in accordance with ASTM D4632.
[3] When seams are required. Values apply to both field and manufactured seams.
[4] The geotextile permeability should be greater than the soil permeability.

The following should be noted for Table 9-7:

- Acceptance of geotextile material shall be based on ASTM D4759.
- Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D4354.
- All values are minimum values; use the value in the weaker principal direction. All numerical values represent minimum average roll values (i.e., test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D4354.

Due to filtration and drainage requirements, woven slit film geotextiles should not be allowed.

Note that as indicated in Figure 9-5, for very poor subgrades (R ≤ 5, CBR ≈≤ 1.5), an even stronger geotextile than Class 1 may be warranted as recommended by Caltrans (2014).

Step 4. Follow the construction recommendations in Section 3.2.1.

3.3.2.2 Stabilization

The requirements for proper performance of geosynthetics used in stabilization applications can be appropriately selected by using the following design steps from FHWA (2008):

Step 1. Identify properties of the subgrade, including CBR, location of groundwater table, AASHTO and/or USCS classification, and sensitivity.

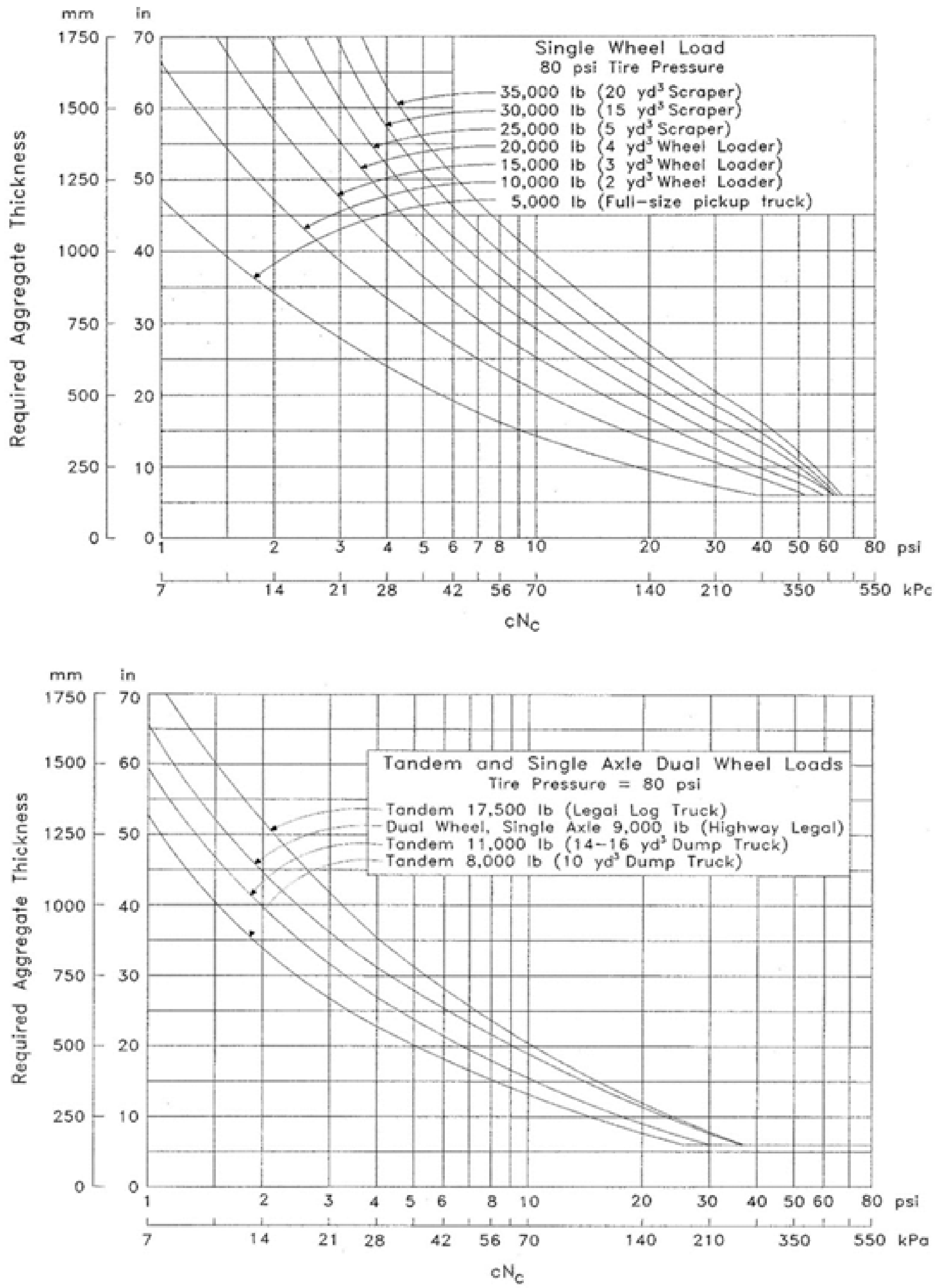
Step 2. Compare these properties to those in Section 3.1.3.1, or with local policies. Determine if a geosynthetic will be required.

Step 3. Design the pavement without consideration of a geosynthetic, using normal pavement structural design procedures.

Step 4. Determine the need for additional imported aggregate to ameliorate mixing at the base/subgrade interface. If such aggregate is required, determine its thickness, t_1, using agencies standard procedures, and reduce the thickness by 50%, considering the use of a geosynthetic.

Step 5. Determine additional aggregate thickness t_2 needed for establishment of a construction platform. The FHWA procedure requires the use of curves for aggregate

thickness versus the expected single tire pressure and the subgrade bearing capacity, as shown in Figure 9-6, which has been developed for highway applications.



Adapted from FHWA 2008, after USDA 1977

Figure 9-6. Stabilization aggregate thickness design curves with geosynthetics for a single wheel loads (top) and for dual wheel loads (bottom).

Note that the charts in Figure 9-6 are the same charts used for determining gravel thickness in Section 2.3.2.1 without geosynthetics). The curves in Figure 9-6 have been correlated with common pavement construction traffic. Select N_c based on allowable subgrade ruts, where:

For Geotextiles

N_c = 5 for a low rut criteria (< 2 inches)

N_c = 5.5 for moderate rutting (2 to 4 inches)

N_c = 6 for large rutting (> 4 inches).

For Geogrids

N_c = 5.8 for geogrids (with a geotextile separator) for low to moderate rutting

FHWA (2008) also contains an alternate method by Giroud and Han (2004a, 2004b), a theoretically based and empirically calibrated design method specifically designed for geogrid-reinforced unpaved roads and areas, which may also be used to obtain t_2. Alternatively, local policies or charts may be used.

Step 6. Select the greater of t_2 or 50% t_1.

Step 7. Check filtration criteria for the geotextile to be used. For geogrids, check the aggregate for filtration compatibility with the subgrade (see Section 2.3), or use a geotextile in combination with the grid to meet the following criteria. The important measures include the apparent opening size (AOS), the permeability (k), and permittivity (ψ) of the geotextile. These values will be compared to a minimum ψ requirement or to the soil properties as follows:

$$AOS \le D_{85\,Subgrade}\,(Wovens) \qquad \text{[Eq. 9-6]}$$

$$AOS \le 1.8 \times D_{85\,Subgrade}\,(Nonwovens) \qquad \text{[Eq. 9-7]}$$

$$k_{Geotextile} > k_{Soil} \text{ and } \psi \ge 0.1\sec^{-1} \qquad \text{[Eq. 9-8]}$$

When using equation 9-7 for noncohesive silts and other highly pumping susceptible soils, a filter bridge may not develop, especially under dynamic, pulsating flow. A conservative (smaller) $AOS \le D_{85subgrade}$ is advised, and laboratory filtration tests are recommended.

Step 8. Determine geotextile or geogrid survival criteria. The design is based on the assumption that the geosynthetic cannot function unless it survives the construction process. For geotextiles, the AASHTO M 288 (AASHTO 2014a) standard categorizes the requirements for the geotextiles based on the survival class (see Table 9-10, which is discussed later in this chapter). For geogrid survivability, the strength requirements and the opening requirements for interlock and separation shown in Table 9-8 have been developed by the FHWA (2008).

The following should be noted for Table 9-8:

- Acceptance of geotextile material shall be based on ASTM D4759.
- Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D4354.
- All values are minimum values; use the value in the weaker principal direction. All numerical values represent minimum average roll values (i.e., test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D4354.

As Table 9-8 shows, for stabilization of soils, the default is Class 1. These requirements may be reduced based on conditions and experience, as detailed in the AASHTO M 288 guidelines for geotextiles and the footnotes in Table 9-8 for geogrids.

Note: As indicated in Figure 9-5, for very poor subgrades ($R \leq 5$, CBR $\approx\leq 1.5$), an even stronger geotextile than Class 1 may be warranted as recommended by Caltrans (2014).

Table 9-8. Geogrid Survivability and Opening Property Requirements for Stabilization and Base Reinforcement Applications

Property	Test Method	Units	Requirement for Geogrid Class 1[1]	Requirement for Geogrid Class 2	Requirement for Geogrid Class 3
Ultimate Multi-Rib Tensile Strength	ASTM D6637	lb/ft (kN/m)	1230 (18)	820 (12)	820 (12)
Junction Strength[5]	GSI GRI GG2	lb (N)	25[2] (110[5])	25 (110)	8 (35)
Ultraviolet Stability	ASTM D4355	%	50% retained strength after 500 hours of exposure	50% retained strength after 500 hours of exposure	50% retained strength after 500 hours of exposure
Aperture Size	Direct measure	in. (mm)	0.5 to 3-inch and Aperture Size ≥ D_{50} and ≤ 2 (D_{85}) of aggregate above geogrid	0.5 to 3-inch and Aperture Size ≥ D_{50} and ≤ 2 (D_{85}) of aggregate above geogrid	0.5 to 3-inch and Aperture Size ≥ D_{50} and ≤ 2 (D_{85}) of aggregate above geogrid
Separation	ASTM D422	mm	D_{15} of aggregate above geogrid < 5 (D_{85}) subgrade Otherwise use separation geotextile with geogrid	D_{15} of aggregate above geogrid < 5 (D_{85}) subgrade Otherwise use separation geotextile with geogrid	D_{15} of aggregate above geogrid < 5 (D_{85}) subgrade Otherwise use separation geotextile with geogrid

Source: FHWA 2008

[1] Default geogrid selection. For stabilization of soils, the default is Class 1. The engineer may specify a Class 2 or 3 geogrid for moderate survivability conditions (e.g., light weight construction equipment), based on one or more of the following:
(a) The Engineer has found the class of geogrid to have sufficient survivability based on field experience.
(b) The Engineer has found the class of geogrid to have sufficient survivability based on laboratory testing and visual inspection of a geogrid sample removed from a field test section constructed under anticipated field conditions (see note 2).

[2] Junction strength requirements have not been fully supported by data, and until such data is established, manufacturers shall submit data from full scale installation damage tests in accordance with ASTM D5818 documenting integrity of junctions. For soft soil applications, a minimum of 6 in. (150 mm) of cover aggregate shall be placed over the geogrid and a loaded dump truck used to traverse the section a minimum number of passes to achieve 4 in. (100 mm) of rutting. A photographic record of the geogrid after exhumation shall be provided, which clearly shows that junctions have not been displaced or otherwise damaged during the installation process.

3.3.3 Primary Design References

- AASHTO. (2014b). Geosynthetic Reinforcement of the Aggregate Base Course of Flexible Pavement Structures – R 50-09(2013). *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 34th Edition, American Association of State Transportation and Highway Officials, Washington, D.C.
- FHWA. (2008). *Geosynthetic Design and Construction Guidelines*. Authors: Holtz, R.D., Christopher, B.R., and Berg, R.R., FHWA-HI-07-092, Federal Highway Administration, U.S. DOT, Washington, D.C., 460p.
- Giroud, J.P. and Han, J. (2004a). Design Method for Geogrid-Reinforced Unpaved Roads – Part I: Theoretical Development. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 130(8), pp. 776-786.
- Giroud, J.P. and Han, J. (2004b). Design Method for Meogrid-Reinforced Unpaved Roads – Part II: Calibration and Verification. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 130(8), pp. 787-797.
- USDA. (1977). *Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads*. Authors: Steward, J., Williamson, R., and Mohney, J., Forest Service, U.S. Department of Agriculture, Portland, OR. Also reprinted as FHWA-TS-78-205.

3.4 Geosynthetics Overview of Construction Specifications and Quality Assurance

3.4.1 Specification Development

Detailed specifications for geosynthetic stabilization layers are included in three key documents: AASHTO M 288 (AASHTO 2014a), AASHTO R 50-09 (AASHTO 2014b), and FHWA-NHI-07-092 (FHWA 2008); and on the *GeoTechTools* website. Each of these provides guide specifications for geotextile and geogrid construction, recommends that the guide specification be modified to suit local conditions and contractors, and provides example specifications. The main considerations include the minimum geosynthetic requirements for design and those obtained from the survivability, retention, and filtration requirements as determined in Section 3.3, as well as the construction requirements covered in Section 3.2 and 3.3. Concerns and criteria for field installation include, for example, the seam lap and sewing requirements, and construction sequencing and quality control. As with other applications, it is very important that an engineer's representative be on site during placement to observe that the correct geosynthetic has been delivered, that the specified construction sequence is being followed in detail, and that no damage to the geotextile is occurring.

3.4.2 Summary of Quality Assurance

Particular attention should be paid to factors that affect geosynthetic survivability: subgrade condition, aggregate placement, lift thickness, equipment operations, and visual observations. Primary quality control considerations will be moisture, density or stiffness, and thickness of the aggregate layer above the geosynthetic. Modern GPS can be used to control thickness; however, point checks should be made to determine the reliability of GPS measurements. The DCP test can also be used to determine the strength and effective depth of the in-place aggregate. Construction equipment moving over the site should be visually observed and rutting should be limited to tolerable design levels, as documented in the specification. Any ruts should be filled in and the contractor should not be allowed to blade the ruts down as this decreases lift thickness. Proofrolling should be performed to evaluate deformation responses and potential for rutting. After the initial granular lift has been placed on the initial portion of the work, a small section should be excavated to expose approximately 1 ft^2 or more of the geosynthetic for visual observation to identify any damage. If damaged, additional sections should be excavated and examined. The contractor should repair the damaged area by either replacement of the geosynthetic or using a patch. The contractor should increase the initial lift thickness and/or lighter ground pressure construction equipment should be used for any remaining construction.

3.4.3 Summary of Instrumentation Monitoring and Construction Control

For stabilization, monitoring will typically consist of measuring rut depth (based on geosynthetic and gravel thickness design requirements). As with aggregate stabilization, if the stabilization layer will be incorporated into the pavement design, then FWD or LFWD tests can be used to determine the equivalent subgrade stiffness value. This would provide a valid equivalent subgrade resilient modulus value that could be used in design, considering that the geotextile will mitigate long-term degradation of the aggregate from subgrade intrusion and pumping over the life of the pavement. As indicated in the gravel section, proofrolling could be performed with instruments for a more accurate assessment of the improved condition. Intelligent compaction methods can be used to confirm the uniformity of the improved layer as well as provide an idea of the composite stiffness achieved for design (see *GeoTechTools* for additional information on intelligent compaction).

To ultimately evaluate pavement performance in terms of stress/strain at the subgrade layer, the geosynthetic can be instrumented with strain gages. This can be performed at the manufacturer's facility, or at another offsite location, such that installation is not disrupting to construction activities. In fact, one geosynthetic can currently be purchased with fiber optic strain gages manufactured into the geosynthetic.

3.5 Geosynthetic Cost Data

3.5.1 Cost Components

The cost components are the geosynthetic material cost (as measured by the square yard), placement, and the additional stabilization aggregate cost (as measured per ton), if required. The equipment used to construct geosynthetic stabilized subgrades and stabilization aggregate is common to highway construction projects, therefore, additional mobilization costs are negligible. However, for extremely soft subgrades that will not support conventional construction equipment, light construction equipment (e.g., wide track, low ground pressure dozers, grade all excavators, partially loaded dump trucks, scrappers, etc.) may be required and will add to the cost of the stabilization (with or without geosynthetics).

The in-place cost of geotextiles used for separation is on the order of $1.00/yd^2. For soft soil stabilization applications, the in-place costs typically range from $1/yd^2 to $2.50/yd^2 for geotextiles and from $1.50 /yd^2 to $4.00/yd^2 for geogrids. As previously mentioned in Section 2.5, in-place compacted gravel materials range from $7 to $20/ton, depending on the specifications, hauling distance and construction conditions.

Geosynthetics are typically a cost-effective alternative to other foundation stabilization methods such as dewatering, demucking, excavation and replacement with select granular materials, utilization of thicker stabilization aggregate layers, or chemical stabilization. For example, when compared to using thicker aggregate alone, the geosynthetic will typically reduce the aggregate thickness by 20 to 40%. For comparison with geosynthetics, hypothetically assuming the in-place gravel has a dry density of 120 lb/ft^3, a six-inch thick layer of aggregate would cost on the order of $1.80 to $5.40/yd^2, a typical savings for a CBR = 2 subgrade. Not only is cost of the gravel saved greater than the typical cost of the geosynthetic, additional cost savings will be achieved from not having to overexcavate and dispose 6 inches of additional subgrade and long-term improved performance of the pavement section is anticipated from using a geosynthetic. Additional cost information on geosynthetics is available on the *GeoTechTools* website.

4.0 MECHANICAL STABILIZATION: LIGHTWEIGHT FILL

An alternate to replacement with aggregate would be to use lightweight fill, including geofoam, foamed concrete, wood fiber, tire shreds, expanded shale and clay, fly ash, boiler slag, air-cooled slag, as well as others. The use of these materials should especially be considered in deeper deposits where shallow surface stabilization may not be effective and thicker granular aggregate, as discussed in Section 2.0, may be effective for control of deformation under wheel load, but would increase the concern for settlement and/or roadway stability. Lightweight fill materials that have been used for geotechnical applications in highway construction are already provided in the Lightweight Fill chapter. This section will specifically focus on use for stabilization of pavements and material already covered in the previous chapter will be referenced.

4.1 Feasibility Considerations

Practically any of the lightweight fill material presented in Chapter 3 would be feasible for improving pavement support conditions, however, caution is advised for materials that could potentially degrade over time (e.g., wood fibers) as that potentially could negatively impact the pavement ride quality and structural integrity over time.

As indicated in the Lightweight Fill chapter, materials are available from the lower end of soil unit weights down to 0.70 lb/ft^3. The lighter end does create some concern for buoyancy and corresponding potential for uplift of the pavement section (see Figure 9-7).

Figure 9-7. Uplift due to flooding of pavement section using lightweight fill.

Therefore, the design must be carefully evaluated for potential uplift in areas of flooding or where the water table can potentially rise into or above the stabilization layer, requiring an anchoring system (e.g., soil fill on top of the lightweight fill) as recommended in the Lightweight Fill chapter or consideration of alternate stabilization measures. Some lightweight fill materials also provide thermal insulation (e.g., geofoam). For these materials, in cold regions, differential icing will be a design concern. The primary issue in using lightweight fills for stabilization will be the pavement support provided by the materials, which will be addressed in the design section below.

4.1.1 Applications

Lightweight fill can be used to reduce settlement and increase stability for pavements constructed over soft subgrades. For deep deposits of compressible soil, the use of lightweight fill should especially be considered. Although more costly than most of the other stabilization methods in terms of construction dollars, these techniques offer immediate improvement, thus accelerating construction. On some projects, the time savings may be more valuable than the construction cost differential.

4.1.2 Advantages and Potential Disadvantages

4.1.2.1 Advantages

Potential advantages in using lightweight fill for subgrade stabilization include:

- Accelerated construction with reduced problems with site accessibility (i.e., may be able to haul in greater quantity of material with light weight equipment).
- Reduced settlement and stability problems for the roadway system.
- Suitability of a wide variety of projects.
- Potential to mitigate frost problems due to thermal insulation properties.
- Potential to create a more uniform support condition (most important for pavements).

4.1.2.2 Potential Disadvantages

The issues identified and discussed as disadvantages in the Lightweight Fill chapter are equally problematic with stabilization applications. These include:

- Availability of fill, which may not be locally or even regionally available, which will of course influence both selection and cost.

- In general, with the exception of locally available recycled materials, lightweight fill stabilization is typically more expensive than other stabilization technologies.
- Environmental concerns with leachate potentially generated by some lightweight fill.
- Durability of fill, which requires protection of some materials (e.g., need to encapsulate some types of fills).
- Geothermal properties which could cause problems with differential icing of pavement surfaces.

4.1.3 Feasibility Evaluations

4.1.3.1 Geotechnical

As with other stabilization technologies, the sensitivity and strength of the subgrade and depth of soft deposits are the key geotechnical parameters. Other important geotechnical parameters include the plasticity of the subgrade that requires improvement (e.g., to identify issues with expansive clays or frost heave). The location of the groundwater table is especially critical for lightweight fill applications as some of these materials will float and several will absorb moisture (i.e., increase in weight), especially if saturated for long periods of time.

Geotechnical characterization of the lightweight fill is important for design purposes. As indicated in the Lightweight Fill chapter, lightweight fills can generally be grouped into two geotechnical categories: materials that behave and have properties similar to granular soils, and materials that have an unconfined compressive strength and behave similarly to cohesive soils in undrained loading. Granular lightweight fill materials include: wood fiber, blast furnace slag, boiler slag, fly ash, expanded shale, clay and slate (ESCS) and tire shreds. Lightweight fill materials with an unconfined compressive strength include geofoam and cellular concrete.

4.1.3.2 Environmental Considerations

The groundwater table is a critical environmental consideration, both for evaluating lightweight fill buoyancy and potential for leachate. Some lightweight fill materials have the potential for groundwater contamination, depletion of oxygen and effects on the pH of groundwater. These effects are listed for each specific lightweight fill in the design section of the Lightweight Fill chapter. Disposal of excavated material is also an environmental consideration.

4.1.3.3 Site Conditions

The most important site characteristics will be requirements for clearing and grubbing, site stratigraphy including subgrade type, strength, stiffness and sensitivity, depth of groundwater table, and topography to accommodate subgrade drainage requirements. For deep soft soil deposits, the potential for large consolidation settlement is also an important consideration in using lightweight fill. On site disposal of excavated material is also a site consideration.

4.1.4 Limitations

Limitations will be related to environmental concerns (i.e., both the impact of the lightweight fill on the environment and the effects of the environment on the lightweight fill), availability of material, cost and long-term performance issues with some lightweight fill materials. However, there are a number of lightweight fill materials to choose from, therefore, it is likely that one of these materials will be suitable for a given project.

4.2 Construction and Materials

4.2.1 Construction

The Lightweight Fill chapter provides good summaries of construction considerations for each of the different lightweight fill materials including methods for placing, compacting where required, and protecting from external elements that could cause durability concerns, where required. For placement, materials in granular form (e.g., slag, fly ash, expanded shale) and chips (e.g., wood and tire shreds) can be placed in layers and compacted. Some materials such as geofoam can be placed in blocks or panels and some materials are placed as foams or slurries using forms. The type of compaction equipment to maximize density while preventing degradation during compaction is provided along with volume reduction for several of the materials due to compaction and shrinkage due to loading from soil cover and pavement.

4.2.2 Materials

An overview of the more common lightweight fill materials that have been used for geotechnical applications in highway construction is provided in the Lightweight Fill chapter presented earlier in this manual. Typical geotechnical engineering parameters that are important for design and construction are also provided in that section as well as additional information on the composition and sources of the lightweight fill materials.

4.3 Design Overview

4.3.1 Design Considerations

The same design procedures used for conventional fills can be used for granular lightweight fill materials with additional considerations dependent on the specific lightweight material. These additional considerations may include evaluation of the durability, water absorption potential, corrosion potential, combustion potential, erosion potential, and environmental impacts. Tables in the Lightweight Fill chapter provide a list of environmental, design, and construction considerations for each of the specific granular lightweight fill materials, as well as summaries of their design parameters. For tire shred embankments, NYSDOT (2015) provides a useful summary of design guidelines that address post-construction compression, separation of tire shreds from surrounding soil, and fill geometry to prevent spontaneous combustion. Design procedures for lightweight fill materials with unconfined compressive strength are unique to the specific material. Design tables are also provided for geofoam and foamed concrete in the Lightweight Fill chapter and comprehensive review of design requirements for geofoam used beneath pavement sections is provided in NCHRP Report 529 *Guideline and Recommended Standard for Geofoam Applications in Highway Embankments* (Stark et al. 2004a) and in the companion NCHRP Web Document 65 (Stark et al. 2004b).

With regard to pavement design, if a minimum of 3 feet of good quality gravel type fill is placed between the pavement structure and the lightweight materials as a cover, then the lightweight material will have little impact on pavement design, even for the more compressible tire and geofoam materials. However, if a thinner cover must be used, the support value for these materials must be determined. For pavement applications, the most important design value will be the resilient modulus of the material. The Lightweight Fill chapter provides ranges of resilient modulus values for a few of the lightweight fill materials. Where resilient modulus is not available, values for CBR and subgrade reaction modulus, also used in pavement design, are provided for some materials; however, correlations of those values with resilient modulus are even more questionable than they are for soils. A summary of the CBR and estimated resilient modulus values reported in the Lightweight Fill chapter as well as FHWA (1997a) is provided in Table 9-9; however, a better approach is to obtain design values from laboratory resilient modulus tests, especially for the granular type materials.

Table 9-9. Pavement Design Parameters for Lightweight Fill

Fill Type	Compacted Density Range (pcf)	CBR	Resilient Modulus, M_R
Geofoam (EPS)	1 - 2	2 to 4	725 – 1450 psi
Foamed Concrete*	20 – 62	15 to 150	$E_{28\ days}$ = 44 – 450 ksi
Wood Fiber	45 – 60	1	1500 psi
Tire Shreds	45 – 60		1/100 Sand
Expanded Shale, Clay and Slate	50 – 65		Similar to gravel
Fly Ash	70 – 90	1 to 15**	Class C increases with time
Boiler Slag	50 - 65	> 250	Similar to or better than gravel
Air-Cooled Slag	70 – 94	> 250	Similar to or better than gravel

* Depends on density, values are estimated from unconfined strengths for 25 to 62 pcf

** Class F & Class C soaked CBR, but if not soaked, Class C will set-up and increase with time (e.g., up to CBR > 100)

Testing is especially important for highly variable lightweight fill typically obtained from recycled materials (e.g., fly ash, wood fibers, tires shreds, slag). The ideal method is to perform field resilient modulus tests on placed material (i.e., on cover soils after placement over the lightweight material), especially for the bulkier materials, such as tire shreds and geofoam. Test sections could be used and a data base could be developed for future design.

Once equivalent properties have been assigned to the lightweight fill, the design of pavement above lightweight fill is similar to standard pavement design procedures. The intent of the pavement design procedure is to provide the most economical thickness and arrangement of pavement materials while providing sufficient reliability to prevent cracking and excessive rutting within the design life of the pavement. It should be noted that a minimum pavement system thickness of 24 inches should be used above most of the lightweight fill materials, other than gravel type lightweight fill that meets durability requirements. For many of the materials, this is due to their relative stiffness. For geofoam, this value is recommended to minimize the potential for differential icing and solar heating.

As an important note, the transition zone between lightweight fill and the roadway embankment soil should be gradual to minimize differential settlement. The calculated settlement gradient within the transition zone should not exceed 1:200 (vertical:horizontal).

4.3.2 Design Steps

As previously indicated, the same design steps used to determine the thickness requirement for conventional granular fills covered in Section 2.3.2 can be used for granular lightweight fill materials. However, Figures 9-1 and 9-6 were developed for aggregates with CBR > 80.

Therefore, aggregate thickness will need to be increased accordingly for lightweight aggregate with CBR < 80 or estimated using an alternate method (e.g. Giroud and Han method in FHWA 2008). In either case, proofrolling should be performed to confirm the adequacy of the stabilization layer thickness.

Granular lightweight fill is also often used for blending with lower quality soils, providing both improved stability and a reduced unit weight. For example, the modulus of tire chips is very low compared to sand, however, a mixture of sand and tire chips (as low as 30% sand according to Edil, 2006) restores the modulus or the mixture to a level comparable to that of sand alone. The blending design methods to determine the required gradation to achieve better stability and/or drainage covered in Section 2.3.2.2 should be used. For pavement design, the primary design parameter for either the granular lightweight fill or the blend will be the resilient modulus to determine the support of the stabilization layer and the equivalent modulus of the stabilized subgrade as covered in Section 4.3.1. Fly ash, especially Class C fly ash, is somewhat unique in that it is cementitious and can be used as an admixture (as covered in Section 6).

For geofoam materials, a comprehensive set of design procedures is provided for both flexible and rigid pavements in NCHRP Report 529 *Guideline and Recommended Standard for Geofoam Applications in Highway Embankments* (Stark et al. 2004a) and in the companion NCHRP Web Document 65 (Stark et al. 2004b). A design catalog for low volume roads is included that can be used to obtain the structural number based on the EPS type, reliability level, and traffic level. After obtaining the structural number, the AASHTO *Guide for Design of Pavement Structures* (AASHTO 1993) or state DOT design manuals can be used to select layer coefficients and determine the most economical pavement system. The procedure also includes a table of minimum recommended AASHTO values for the thickness of the asphalt concrete and aggregate base based on traffic Equivalent Single-Axle Loads (ESALs). For rigid pavement design, a set of design catalogs is used to obtain the rigid concrete thickness based on the EPS type, reliability level, ESALs, modulus of rupture, and whether or not edge support and/or a load transfer device is included in the design. The design catalogs are based on the same assumptions and general procedures as those from the AASHTO *Guide for Design of Pavement Structures*. For preliminary estimation of the dead load imposed by the pavement system, a thickness of 24 inches and unit weight of 130 pcf should be used, according to the NCHRP 529 report.

4.3.3 Primary Design References

- FHWA. (2010). *Geotechnical Aspects of Pavements*. Authors: Christopher, B.R., Schwartz, C., and Boudreau, R., FHWA NHI-10-092, Federal Highway Administration, U.S. DOT, Washington, D.C., 568p.

- NYSDOT. (2015). Guidelines for Project Selection, Design, and Construction of Tire Shreds in Embankments. *Geotechnical Engineering Manual*, GEM-20 Revision #3, New York State Department of Transportation, Albany, NY.
- Stark, T.D., Arellano, D., Horvath, J.S., and Leshchinsky, D. (2004a). Guideline and Recommended Standard for Geofoam Applications in Highway Embankments. *NCHRP Report 529* (Project 24-11), National Cooperative Highway Research Program, Transportation Research Board, The National Academies, Washington, D.C.
- Stark, T.D., Arellano, D., Horvath, J.S., and Leshchinsky, D. (2004b). Geofoam Applications in the Design and Construction of Highway Embankments. *NCHRP Web Document 65* (Project 24-11), National Cooperative Highway Research Program, Transportation Research Board, The National Academies, Washington, D.C.

4.4 Overview of Construction Specifications and Quality Assurance

Specifications for lightweight fill materials are well covered in the in the Lightweight Fill chapter including quality assurance requirements and monitoring for construction control. Typical specifications included in that chapter should be modified to emphasize that the provided materials shall meet the specified resilient modulus and durability requirements for pavement support. Performance monitoring for pavement applications could include FWD or LFWD testing to confirm design compliance as well as obtain data for future design. Specifications for lightweight fills are available on the *GeoTechTools* website.

4.5 Cost Data

Cost data is provided in the Lightweight Fill chapter and is equally applicable to lightweight fill used for pavement subgrade stabilization. Additional cost information on lightweight fills is available on the *GeoTechTools* website.

5.0 MECHANICAL STABILIZATION: RECYCLED MATERIALS

As indicated in the Introduction, there are two forms of recycled materials used for subgrade stabilization in pavements: (1) reuse of the pavement materials themselves, and (2) the use of recycled waste materials for subgrade stabilization or as a substitute for aggregate. The first type represents the main use of recycled materials for pavement construction, and consists of recycled concrete pavement (RCP) or reclaimed asphalt pavement (RAP) from the existing pavement at the site or hauled in from other construction sites. The second type of recycling involves a number of fill materials, most of which are lightweight materials (i.e., slag, fly ash, bottom ash, tire shreds, wood chips) that were covered in Section 4, and will not be included in this section. Two recycled materials that were not discussed in the previous section include foundry sand, which will be reviewed in this section, and recycled concrete materials (RCM) from both pavement (i.e., RCP) and other non-pavement engineering projects.

There are several asphalt pavement recycling techniques, such as hot mix recycling, hot in-place recycling, cold mix recycling, cold in-place recycling, and full depth reclamation, which have evolved over the past 35 years. However, only the removal and reuse of the asphalt as an unbound aggregate material used for stabilization of the subgrade will be reviewed in this section. The in-place recycling methods involve adding foamed asphalt or asphalt emulsion as a binder and using the bound material as base or subbase within the pavement section. RAP is usually produced by milling and collecting the old asphalt for reuse, with aggregate gradations achieved by pulverization of the collected material.

There are also three methods of slab fracturing techniques used to rehabilitate concrete pavements: crack and seat, break and seat, and rubblization. All three are typically used in-place as base course layers and are again not considered for subgrade stabilization. In fact, poor subgrade conditions often exclude the use of these methods. Crushing of the concrete pavement (or other waste concrete material) to typical aggregate gradations for use in road construction (i.e., sand and gravel sized particles) is used to produce RCP. Only applications where RCP is used as aggregate to stabilize subgrades are considered in this section. As previously indicated, RCP will often be combined with other concrete recycled materials from other concrete structures as they are basically the same and referred to in the rest of this section as RCM.

Additional information on recycled materials used in pavement systems can be found in FHWA-RD-97-148 *User Guidelines for Waste and Byproduct Materials in Pavement Construction* (FHWA 1997a).

5.1 Feasibility Considerations

5.1.1 Applications

In subgrade stabilization application, pavement materials are recycled on site or hauled to the site from another construction site to provide a stabilization layer where subgrade improvement is required for better support of the pavement section. RAP and RCM may also provide an improved base course layer, resulting in better support over marginal subgrade conditions. For review of material and design considerations for base/subbase applications, interested readers are referred to FHWA (1997a).

5.1.2 Advantages and Potential Disadvantages

5.1.2.1 Advantages

Using RAP or RCM can reduce pavement reconstruction cost, reduce the cost of disposing of waste, and conserve the diminishing supply of aggregates. When properly designed and constructed, pavements built with recycled materials have performed as well as pavements built with natural gravel materials.

5.1.2.2 Potential Disadvantages

There are several potential disadvantages with the technology including:

- Material is less uniform than quarried aggregates, especially considering that it may be commingled from different project sources, but with good quality control, this is usually not a problem for stabilization applications.
- RAP could be commingled with contaminated (i.e., poor quality) base/subbase material due to over grinding or overzealous excavation.
- 100% RAP has a much lower CBR than gravel.
- Residual oil and contamination may lead to environmental issues. Some agencies require that a minimum separation be maintained between watercourses and RAP or RCM.
- For crushed concrete being used as aggregates, there is a potential for “tufa” like precipitates (a white pasty substance developed from unhydrated cement) to leach from recycled concrete material and create a concern for clogging of drainage systems adjacent to the stabilized subgrade section.
- RCM is susceptible to some freeze/thaw degradation. Freeze thaw durability tests should be performed if the stabilization layer is within the anticipated frost zone.

- Due to its high alkalinity, recycled concrete in contact with aluminum or galvanized steel pipes can cause corrosion in the presence of moisture. RCM may also contain chloride ions from the application of deicing salts making them even more corrosive.

5.1.3 Feasibility Evaluations

5.1.3.1 Geotechnical

The geotechnical issues related to using recycled material as aggregate for stabilization, either as thicker gravel or with geosynthetics are practically the same as covered in those sections. The location of groundwater and location of nearby water bodies should be identified in the geotechnical investigation for environmental considerations.

For recycling the pavement on a project, the type of subgrade soil, the quality of the in-place base and subbase (i.e., subgrade contamination level) and groundwater issues are key geotechnical issues in the feasibility evaluation. The type of pavement recycling method will depend on the strength of the subgrade soil, which must be adequate to support the grinding or rubblization equipment. To improve the feasibility of recycling the pavement system, drains should be installed and the site dewatered before in-place recycling if wet, saturated soil conditions are present.

5.1.3.1 Environmental Considerations

The primary environmental considerations for recycling are weather related as many of the processes use water, so cold temperature is a concern. Also, exposing the subgrade to wet weather when rubblizing or cracking concrete could create significant disturbance or even unstable conditions due to increase in pore water pressure in the subgrade under dynamic impact. Noise from vibration and impact must also be considered in using this technology, especially in urban areas that have noise restrictions. The effect of vibration and/or impact on nearby buildings and other structures must also be considered. Dust control may also be an issue.

The location of the groundwater and runoff control are important considerations, depending on the type and potential for contamination from the recycled material, as indicated in the Potential Disadvantages section.

5.1.3.2 Site Conditions

For recycling pavements, to determine the feasibility, type and method or recycling, the key elements of the evaluation are (from TRB Transportation Research Circular E-C087 *Rubblization of Portland Cement Concrete Pavements* [TRB 2006]):

- Perform a distress survey of the existing pavement
    - Cracking
    - Joint deficiencies
    - Surface defects
    - Miscellaneous distresses
- Evaluate existing pavement structure
    - Layer types
    - Layer thickness
    - Shoulder condition
- Determine soil conditions
    - Soil types
    - Bearing value (e.g., CBR or M_r)
    - Moisture condition

For subgrade stabilization, site condition requirements are similar to thicker gravel stabilization layers, including: clearing and grubbing requirements; location of the water table and subgrade drainage requirements; and, subgrade type, strength, stiffness and sensitivity. One of the advantages in using a geosynthetic is that disturbance of the subgrade is better controlled as discussed in the construction section. On site disposal of excavated material is also a consideration. Deep soft soil deposits, the potential for large consolidation settlement is an important factor as well. Also, if recycled materials are to be blended with the subgrade or other granular materials, the site condition considerations in the thicker gravel section should be reviewed.

5.1.4 Limitations

During in-place recycling, the fractured pavement section must be adequate to support multiple passes of the equipment. In-place recycling may not be feasible where subgrade soils are very soft (e.g., CBR ≤ 2 based on the experience of several DOTs) or highly sensitive. High water level should be addressed before in-place recycling is implemented.

5.2 Construction and Materials

5.2.1 Construction

Construction follows the same practice as reviewed in the thick granular layer section. Soft ground construction equipment (e.g., low ground pressure wide track dozers, extended backhoe and excavators, partially loaded dump trucks, etc.) should be considered for excavation and fill placement to avoid disturbance of the subgrade. In sensitive soils, such disturbance can lead to significant overuse of gravel or a section that does not perform as well as anticipated. Any equipment that results in subgrade or granular layer rutting of more than a few inches should not be allowed to operate on the site.

Underdrain systems (either installed or existing) should be functioning, especially if the existing pavement is to be recycled and weak subgrade and/or high water table are anticipated in the recycling zone. In order to provide adequate time for improvement of the subgrade conditions, the drainage system should be installed and functioning as far in advance of the recycling process as possible. The drainage system will also provide drainage of the pavement section during rain events.

5.2.2 Materials

Gradation and density of the recycled material must be controlled for all methods of recycling. For RCM, any reinforcing steel must be removed before the RCM is crushed and screened to meet gradation requirements. Each recycling method has specific maximum size requirements. Uncrushed materials (oversized pieces of asphalt or concrete) should not be placed, as they may impact future construction activities (e.g., create difficulty with uniform compaction or even voids). If composite pavements are recycled, The RAP content in the RCM should be limited to 20 percent to prevent a reduction in bearing strength and stiffness (FHWA 1997a).

The amount of fines (i.e., smaller than a #200 US sieve) must also be controlled with consideration for drainage potential. Recycled aggregates may also require washing to remove fines (i.e., for free draining materials), dust, impurities, or contaminants. For example, washing RCM is often required to remove dust as a measure to reduce tufa formation and to remove road salts. To control tufa precipitate formation, only RCM that does not contain significant quantities of unhydrated cement or free lime should be used.

If different stockpiles are used, tests (e.g., gradation, moisture, density and leach tests) should be performed on each stockpile to determine the consistence and quality of the materials. The same compaction requirements for natural materials should be used for recycled aggregates using vibratory roller compaction methods. Subgrade strengths must be checked before and

after recycling as the recycling process may reduce the strength of the subgrade soil. The amount of fines from subgrade intrusion or contaminated base materials must be minimized as part of the construction process.

5.3 Design Overview

The design requirements for recycled asphalt (RAP) and recycled concrete (RCM) aggregates are essentially the same as natural aggregates and the materials must be evaluated with respect to the same property requirements as the material they will replace. The pavement support values (e.g., resilient modulus of the material) must be determined and an assessment be made of the durability and drainage characteristics based on laboratory tests. The pavement support value should be determined based on lab tests. Field trials using FWD to confirm the as constructed properties are also recommended. Durability is a critical issue with many of these materials, and, obviously, an assessment of environmental issues must be made. Additional considerations may include evaluation of the durability, corrosion potential (especially for RCM) and environmental impacts.

5.3.1 Design Considerations

Design of the pavement depends on assessment of the improvement provided by the recycled materials. For hauled-in RCM, RAP, and foundry sand, the materials can be treated as aggregate, tests can be performed in the laboratory before placement to obtain design properties, and construction quality control and quality assurance can target meeting or exceeding those property requirements. However, for recycling of existing pavements, design must either rely on sending material to the lab during the crushing process or on sampling and testing in the field. In situ testing after placement and/or test pits for direct sampling and testing must be performed. As indicated in the construction section, a thorough investigation of the subgrade soils must be performed to determine whether or not the pavement can be successfully rubblized.

5.3.1.1 Recycled Asphalt

With the sizing, RAP can often only be effectively screened down to a maximum size of 2 inches. If a significant amount of contaminated base course (i.e., containing significant amount of fines) is removed with the asphalt, the hydraulic properties of the aggregate could also be poor.

The CBR range for RAP is typically on the order of 20 to 25 percent and the permanent deformation should be anticipated to be much greater than natural aggregates, on the order of 10 times greater plastic strains (Edil 2011). RAP can be blended with natural aggregates,

which will have a significant strengthening effect over time (e.g., aggregates containing 40% RAP have been found to produce CBR values exceeding 150 after 1 week (FHWA 1997a)).

5.3.1.2 Recycled Concrete

The design requirements for RCM in embankment construction are the same as those for conventional aggregates. RCM is highly angular in shape and CBR values ranging from 90 to more than 140 percent (depending on the angularity of the virgin concrete aggregate and strength of the Portland cement matrix), which is comparable to crushed limestone aggregates. The high alkalinity of RCM (pH greater than 11) can result in corrosion to aluminum or galvanized steel pipes in direct contact with RCM and in the presence of moisture, which is mainly a design consideration for utilities, guard rail posts, etc.

Edgedrains and deep drains that will extend through or adjacent to the stabilization layer should be designed considering the potential formation of tufa precipitate. Open graded gravel should be placed adjacent to the stabilization material with no geotextile filter across the interface in that portion of the drain. The gravel should be designed as a filter for the RCM. Geotextiles can be used beneath or above the stabilization layer and wrapped around the remainder of the drain.

5.3.1.3 Waste Recycled Material

Other recycled materials can also be used as a replacement for natural gravel materials for stabilization of base (e.g., foundry sand and many of the materials reviewed in the lightweight fill section) and, in some cases (e.g., glass and tire shreds) drainage aggregate. As indicated above, the materials must be evaluated with respect to the same property requirements as the material they will replace. Durability is a critical issue with many of these materials, and, obviously, an assessment of environmental issues must be made.

Foundry sand is the excess waste product that results from using sand, that is bonded by clays (e.g., bentonite) or chemical elements (e.g., phenolic urethane), to form molds for metal casting and in cores that form the internal shapes and cavities within the casting. Other carbon additives (e.g., coal dust) are used to control gas permeability, strength, and other properties of the mixture (for more information see Edil 2006).

Foundry sand has been found to have CBR values of approximately 4 to 20 and M_r values of approximately 6,000 psi to 30,000 psi. These values are less than those for gravel used for base course layers but are generally equal or greater in strength and stiffness for subbase material, therefore, foundry sand is a very good candidate for aggregate used for subgrade stabilization (Edil 2006). Strict moisture control is required for compaction due to the presence of bentonite, which will vary between foundries and maybe within the stockpile.

However, the presence of high bentonite content (e.g., above 6 %) may make the compacted materials rather impermeable, and it could be used to provide a partial encapsulation cover to prevent moisture from migrating into moisture sensitive soils beneath the foundry sand. Permeability tests should be performed for these types of applications.

5.3.2 Design Steps

If the recycled material is simply replacing aggregate for subgrade stabilization, the design steps are the same as in Section 2.3.2, and if geotextiles are being used with the recycled materials, the design should follow the steps outlined in Section 3.3.2. However, if Figures 9-2 or 9-7 are used, an increased thickness for the stabilization layer will be required if recycled material has a CBR < 80. Alternatively, the Giroud and Han method in FHWA (2008) could be used as the CBR of the material is an input for the design model. In either case, proof rolling should be performed to confirm the adequacy of the stabilization layer thickness.

For in-place recycling, step by step procedures are provided by FHWA 1997b).

5.3.3 Primary Design References

- FHWA. (1997a). *User Guidelines for Waste and Byproduct Materials in Pavement Construction*. Authors: Chesner, W.H., Collins, R.J., and MacKay, M.H., FHWA-RD-97-148, Federal Highway Administration, U.S. DOT, Washington D.C., 683p.
- Asphalt Recycling and Reclaiming Association (ARRA). (2001). *Basic Asphalt Recycling Manual*, Annapolis, MD, http://www.cdrecycling.org/assets/concrete-recycling/1-124-barm1.pdf.
- FHWA. (1997b). *Pavement Recycling Guidelines for State and Local Governments: Participant's Reference Book*. Authors: Kandhal, P.S. and Mallick, R.B., FHWA-SA-98-042, Federal Highway Administration, U.S. DOT, Washington, D.C.

5.4 Overview of Construction Specifications and Quality Assurance

5.4.1 Specification Development

For recycled asphalt (RAP) and recycled concrete (RCP) aggregates again are essentially the same as natural aggregates and most state DOTs have specifications that cover the material, placement, and compaction requirements. FHWA (1997a) contains a “Users Guideline” section for RAP, RCM and other recycled waste materials used as fill including processing requirements, engineering properties, design considerations, construction procedure including special considerations such as cold weather and corrosion mitigation, and

unresolved issues, all of which should be considered in the preparation of specifications. This document as well as specific state DOT specifications is reviewed in *GeoTechTools*.

The Recycled Materials Resource Center at the University of Wisconsin-Madison has published two detailed specifications for consideration when using RAP and RCP materials as unbound road base (Edil 2011), which would also be appropriate for maintaining good control over RAP and RCP aggregates used for stabilization. These include:

- Standard Specification for Grading Requirements and Density Determination of Recycled Asphalt Pavement Materials as Unbound Base and Subbase for Highways and Airports (in ASTM balloting process).
- Standard Guide for Recycled Aggregates As Unbound Roadbase (in draft form).

The standard guide is focused on requirements for the operators' control of the recycled material and crushing of the pavement materials. The operator is assumed to:

1. Secure a supply of disposed asphalt and concrete, stockpile it, process/crush the disposed material, test and stockpile the recycled aggregate product.
2. Be compliant with local and state jurisdictions, solid waste management rules, laws, and regulations.
3. Be compliant with air quality and other local, state and federal rules, laws and regulations.
4. Have the necessary plans in place for protecting workers' health, safety and the environment.
5. Meet recycled aggregate material quality standards specified to help ensure optimum performance when used in the construction of roads, highways and foundations.

5.4.2 Summary of Quality Assurance

As with gravel, primary quality control considerations will be moisture, density or stiffness, and thickness. Since each source of recycled material will be different, it is important to take random samples of the stockpiled material. Stockpiling and handling should be monitored for potential segregation.

For stabilization, modern GPS methods can be used to control thickness, however, point checks should be made to determine the reliability of GPS measurements. The DCP test can also be used to determine the strength and effective depth of the in-place aggregate. Proof rolling should be performed to evaluate deformation responses and potential for rutting.

5.4.3 Summary of Instrumentation Monitoring and Construction Control

The FWD test applied from the surface of the improved layer provides good performance measurements of both stiffness of that layer and stiffness of the subgrade after in place recycling. Evaluation of the resilient modulus from the FWD results can be used to confirm that the process meets the design requirements. This value also provides a reference point for future FWD on the new pavement surface for monitoring long-term performance.

5.5 Cost Data

For stabilization applications, recycled pavement materials are typically incorporated into a highway reconstruction project, either as aggregate component of subbase/base course(s) or road bed embankment materials.

5.5.1 Cost Components

In general, specifications should allow the use of recycled materials, but not force their use. This allows market forces to determine the most cost effective and sustainable use of recycled materials. Both the availability of alternative materials and the demand from other markets for recycled materials have a large influence on project cost savings that are passed on to the owner and make them difficult to quantify. Except for very large projects where the potential cost savings may be significant (greater than 100,000 tons of available recycled material), preliminary project budgets should include a nominal cost savings of $2.00 to $5.00 per ton of recycled material that is incorporated onsite. Additional cost information on recycled pavement materials is available on the *GeoTechTools* system.

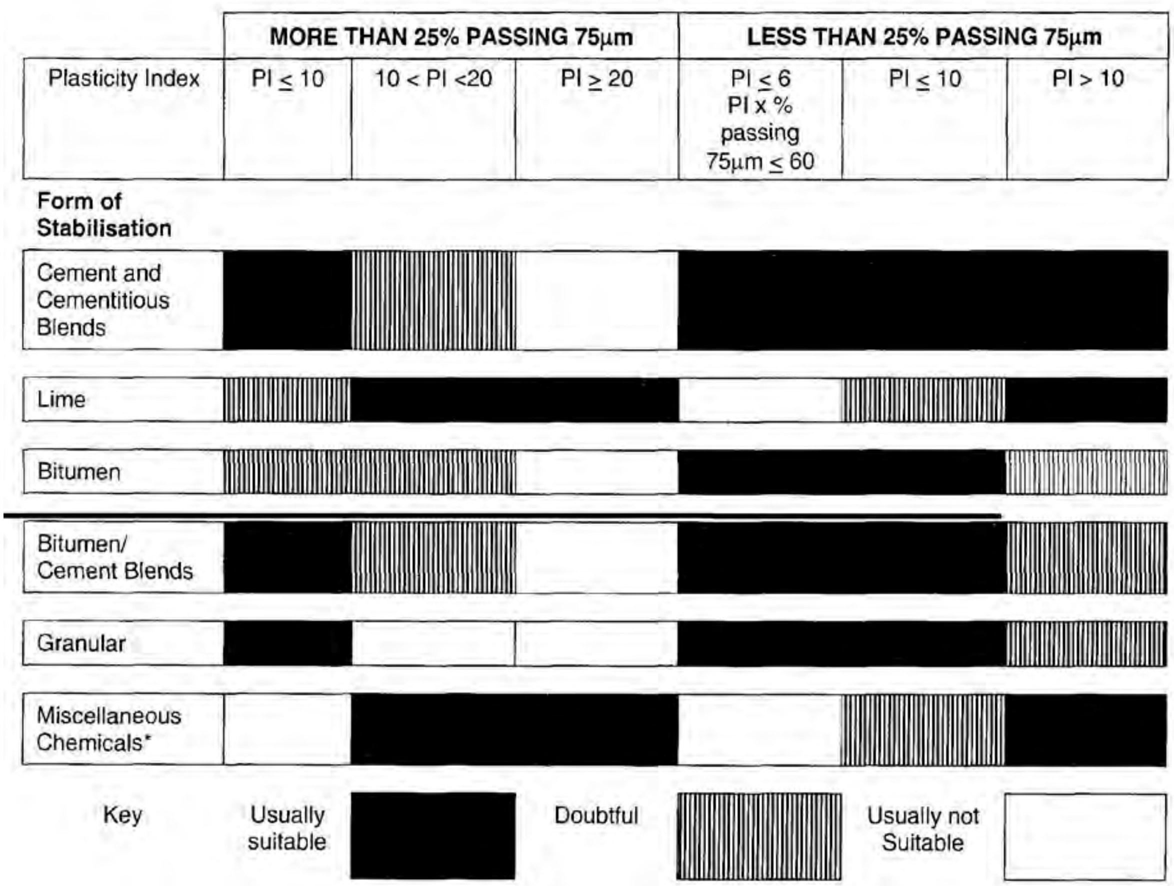
6.0 ADMIXTURE/CHEMICAL STABILIZATION

As previously indicated in the Section 1.0 Introduction, there are a variety of admixtures that can be mixed with the subgrade to improve its performance. This section contains a general overview of each admixture, followed by a generalized outline for determining the optimum admixture content requirements. References are provided for the design details for each specific method.

6.1 Feasibility Considerations

6.1.1 Applications

The various admixture types are shown in Figure 9-8, along with initial guidance for evaluating the appropriate application of these methods.



The forms of stabilization may be used in combination, e.g., lime stabilization to dry out materials and reduce their plasticity, making them suitable for other methods of stabilization.
Miscellaneous Chemicals* should be taken as a broad guideline only. Refer to trade literature for further information.

Austroads 1998

Figure 9-8. Guide for selection of admixture stabilization method(s).

Admixtures can be used to stabilize soft and/or compressible soils, expansive soils, and wet soils. Cementing agents, such as Portland cement, bitumen, lime, cement kiln dust (CKD) and lime-fly ash, effectively bond individual soil particles together, and also act to partially remove capillary passages, thereby reducing the potential for moisture movement. Care must be taken when using lime and lime-fly ash mixtures with clay soils in seasonal frost areas. Stabilization, as used for expansive soils, refers to the treatment of a soil with such agents as bitumen, Portland cement, slaked or hydrated lime, and fly ash to limit its volume change characteristics. Chemical stabilization can substantially increase the strength of the treated material.

6.1.2 Advantages and Potential Disadvantages

6.1.2.1 Advantages

There are several advantages of using admixtures:

- This is an in-place treatment method, eliminating over excavated spoil including disposal and replacement costs.
- Admixtures increase the stiffness of subgrades.
- Admixtures can be used to reduce soil plasticity, water absorption, swelling potential, and compressibility and increase soil strength and modulus with time.
- Lime will reduce swell in an expansive soil to greater or lesser degrees, depending on the activity of the clay minerals present.
- There are a number of admixtures that have applicability to most soil types.

6.1.2.2 Potential Disadvantages

Admixtures, in most cases, require a curing time of several days and often up to one week. Adequate curing is important if the strength characteristics of the soil are to be improved.

Another downside of admixtures is that they require up front lab testing to confirm their performance and very good field control to obtain a uniform, long lasting product, as outlined later in this section. There are also issues of dust control and weather dependency with some methods that should be carefully considered in the selection process.

Some admixture (e.g., lime or fly ash) treatment of soil can convert a soil that shows negligible to moderate frost heave into a soil that is highly susceptible to frost heave, acquiring characteristics more typically associated with silts. In the case of lime, it has been reported that this adverse effect was been caused by an insufficient curing period. For lime,

CKD, and possibly cement, sulfates present in the soil could potentially react with free lime and form expansive minerals, resulting in additional swelling where none previously existed.

6.1.3 Feasibility Evaluations

6.1.3.1 Geotechnical

The applicability of each admixture for specific geotechnical characteristics are provided in Table 9-2 and Figure 9-8. For soils to be stabilized with cement, proper mixing requires the soil to have a PI of less than 20 and a minimum of 45% passing the No. 40 sieve. However, highly plastic clays that have been pretreated with lime or fly ash are sometimes suitable for subsequent treatment with Portland cement. For cement stabilization of granular and/or nonplastic soils, the cement content should be 3 to 10% of the dry weight of the soil, and the cured material should have an unconfined compressive strength of at least 150 psi within 7 days. The Portland cement should meet the minimum requirements of AASHTO M 85. The cement-stabilized subgrade should be compacted to a minimum density of 95% as defined by AASHTO T 134.

For lime stabilization of clay, or highly plastic clayey soils, the lime content should be from 3 to 8% of the dry weight of the soil, and the cured mass should have an unconfined compressive strength of at least 50 psi within 28 days. The optimum lime content should be determined with the use of unconfined compressive strength and the Atterberg limits tests on laboratory lime-soil mixtures molded at varying percentages of lime. As discussed later in this section, pH can be used to determine the initial, near optimum lime content value. The pozzolanic strength gain in clay soils depends on the specific chemistry of the soil, e.g., whether it can provide sufficient silica and alumina minerals to support the pozzolanic reactions. Plasticity is a rough indicator of reactivity. A plasticity index of about 10 is commonly taken as the lower limit for suitability of inorganic clays for lime stabilization. The lime-stabilized subgrade layer should be compacted to a minimum density of 95%, as defined by AASHTO T 99.

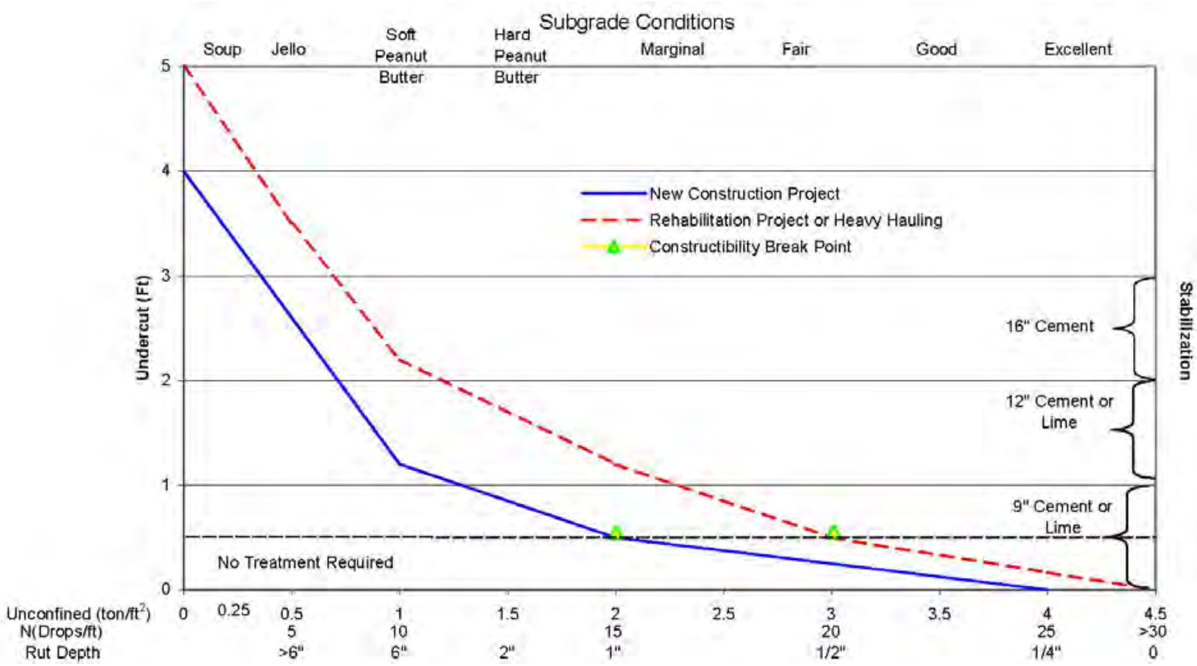
Pozzolanic fly ash is suitable for cohesive and granular materials. Asphalt stabilization is suitable for silty, sandy and granular materials.

6.1.3.2 Environmental Considerations

As indicated under the disadvantages, dust control and weather are important environmental considerations for selection of these technologies.

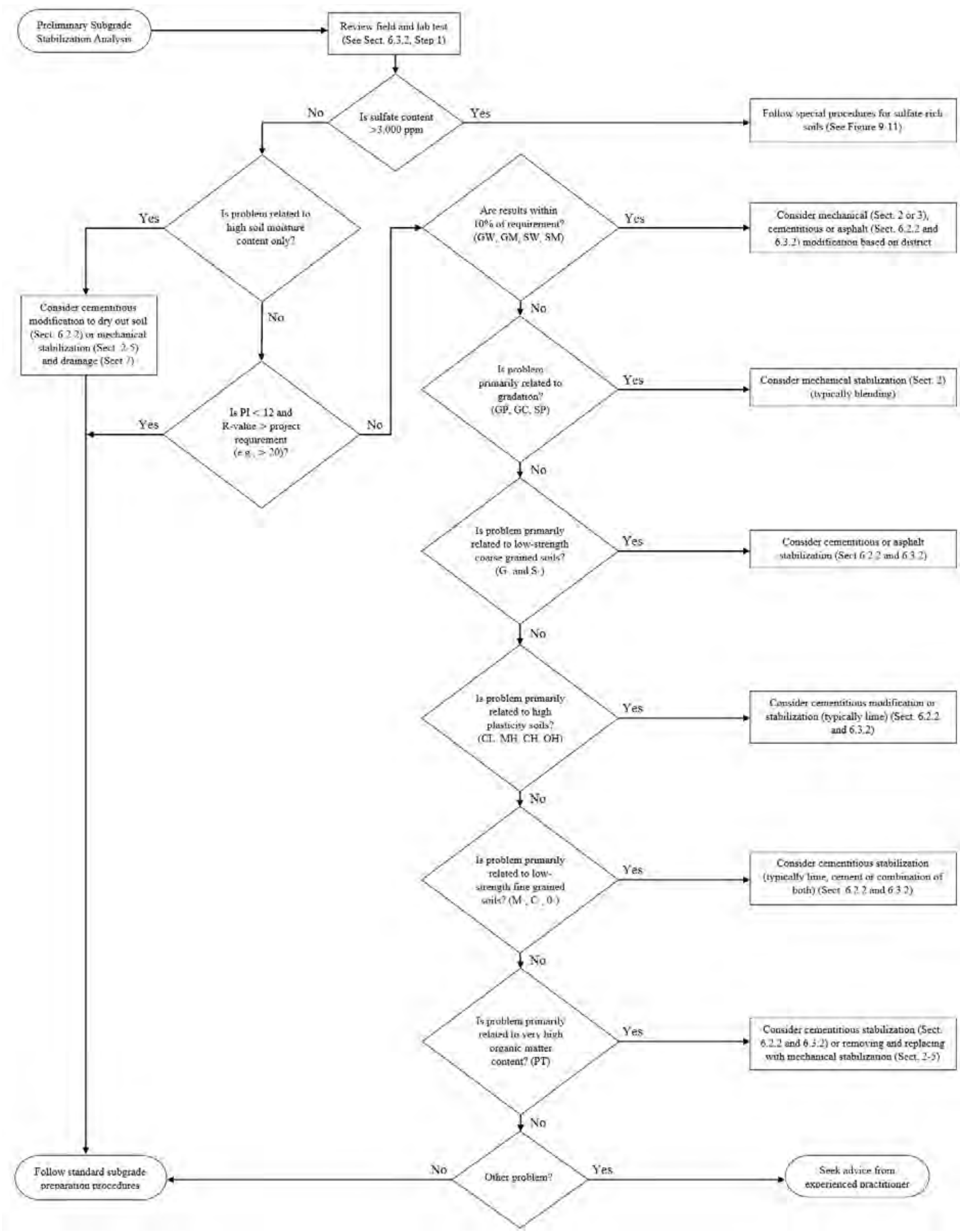
6.1.3.3 Site Conditions

The graph in Figure 9-9 and the flow charts in Figures 9-10 and 9-11 provide a summary of site conditions that two agencies (Ohio DOT and Caltrans) use to evaluate selection of appropriate admixture strategies.



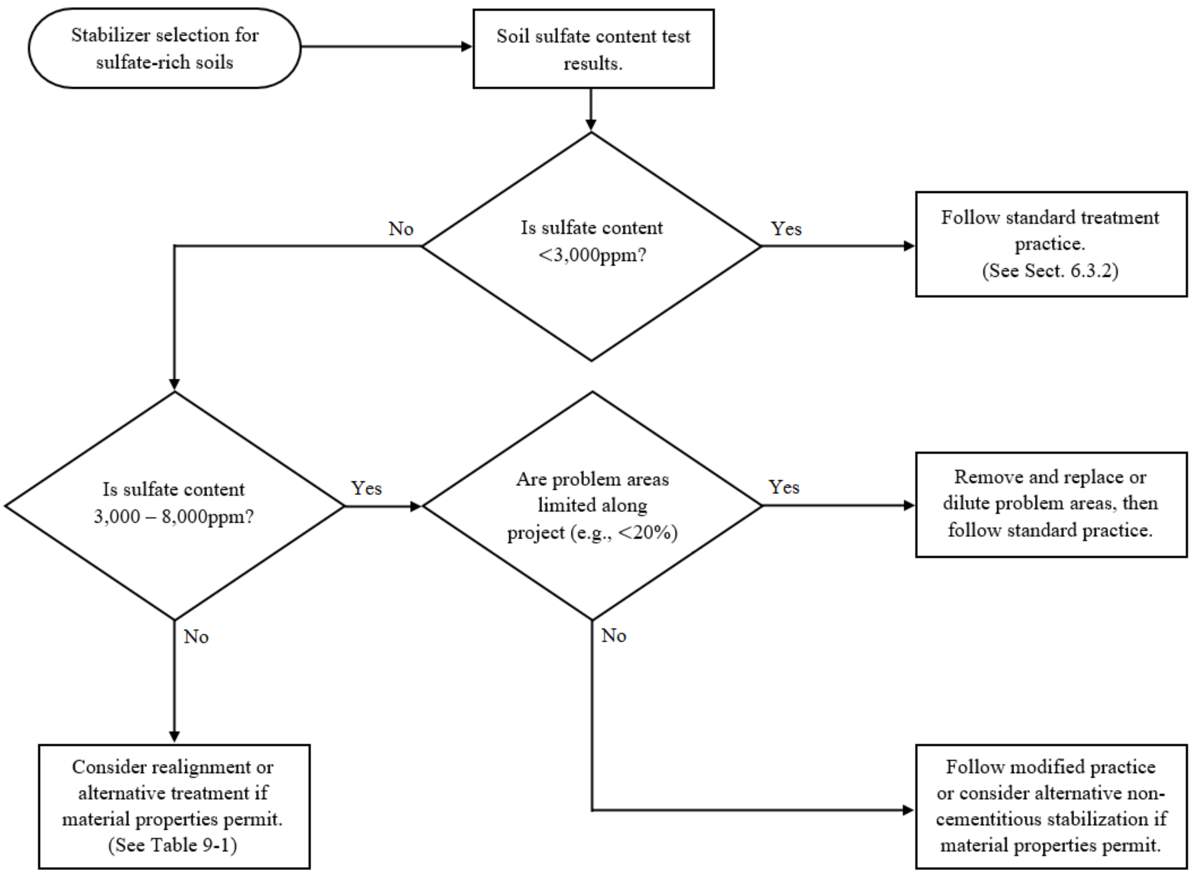
Ohio DOT 2002

Figure 9-9. Site conditions for considering cement or lime stabilization.



After Jones et al. 2010

Figure 9-10. Selecting a first-level project subgrade stabilization strategy.



After Jones et al. 2010

Figure 9-11. Selection of stabilization strategy for sulfate-rich soils.

As can be seen from the flowcharts in Figures 9-10 and 9-11, along with Table 9-2 and Figure 9-8, site conditions to be evaluated include classification, moisture content, plasticity, sulfate content, organic content and strength of soils to be stabilized. The location of the water table should also be identified as high water levels may impede construction and require dewatering for these techniques to be feasible.

6.1.4 Limitations

One of the main limitations is soils that are too soft to support the mixing stabilization equipment. As indicated in Section 1 and Figure 9-8, if the PI is less than 12 and the R > 20, stabilization is generally not required. However, if the CBR < 1 (R < 1), then the subgrade will likely not support the equipment required to mix in the admixture, and other technologies (e.g., geosynthetics and/or lightweight fills) should be considered. The other primary limitation is the presence of sulfates in the subgrade. Cementitious and pozzolanic stabilization should not be used when the sulfate content of the subgrade soil exceeds 8000 ppm and as indicated in Figure 9-10, and special stabilization strategies are required at sulfate contents greater than 3000 ppm, as shown in Figure 9-11 and swell tests should be

performed. The presence of organics will also reduce the effectiveness of cementitious and pozzolanic stabilization methods requiring significant increases in quantities, if they will work at all.

6.1.5 Alternative Solutions (or Technologies)

The use of more gravel, geosynthetics and gravel, and/or dewatering are standard alternatives to using admixtures for stabilization of weak soils. Geosynthetics and lightweight fill are especially effective where soils are too soft to use admixtures. For swelling soils, partial encapsulation could be considered as covered in Section 8 Moisture Control.

6.2 Construction and Materials

6.2.1 Construction

The basic construction steps for chemical admixture stabilization of subgrade soils are (1) pozzolan delivery and distribution, (2) mixing, (3) compacting, and (4) curing. Pozzolans can be applied to a soil either dry or as a slurry. In the case of dry lime, the lime may be either in the form of dry hydrated lime, which is very fine-grained and, thus, may pose dust control problems, or dry quicklime, which is granular and much less dusty.

The pozzolanic material specified for stabilization or modification is distributed along the road alignment, either via bags that are spread manually, by pneumatic trucks with spreader bars, or by dump trucks with controlled tailgate openings. Lime slurries can be mixed in a central mixing plant or in various types of portable mixing systems. A typical lime slurry mixture would consist of 1 ton of lime mixed with 500 gallons of water to produce 600 tons of slurry with 31% lime solids (Transportation Research Board 1987).

Adequate mixing of the pozzolanic material with the soil is critical; poor mixing is the leading cause of unsatisfactory stabilization results. Subgrade soils can be mixed with the pozzolan on site by disking, repeated blading, or by traveling rotary or pug-mill mixing equipment.

Mixing is usually done in thin lifts and often with multiple passes, with the lift thickness and number of passes dependent upon the soil type and the mixing equipment being used. A two-stage mixing process is sometimes used for highly plastic materials; the reduced plasticity and coarser texture that develops during curing for several days after the initial mixing makes the soil more workable for final mixing and compaction.

Compaction of chemically stabilized soil mixtures follows standard procedures. However, with respect to lime stabilization, the addition of lime will generally decrease the maximum

density and increase the optimum water content at a given compaction energy, which may cause problems determining the percentage of specified density achieved by the field compaction. Compaction curves of the in situ lime-soil mixture at the time of compaction may be required to determine the appropriate density values for field compaction control.

Curing at temperatures above 40°F and with adequate moisture is essential for the pozzolanic reactions underlying the long-term strength gains in lime-stabilized soils. A cure period of 3 to 7 days is typically employed, with adequate moisture maintained either through moist curing (e.g., truck sprinklers) or by applying an asphalt seal over the surface.

Similarly, modified soil (lime, cement and/or fly ash) will require special quality assurance considerations. Final evaluation of the stabilized subgrade surface should be made by proof rolling, DCP, and/or FWD.

6.2.2 Materials

6.2.2.1 Portland Cement and Cement Kiln Dust (CKD)

Several different types of cement have been used successfully for stabilization of soils. Type I normal Portland cement and Type IA air-entraining cements were used extensively in the past, and produced about the same results. At the present time, Type II cement has largely replaced Type I cement as greater sulfate resistance is obtained, while the cost is often the same. High early strength cement (Type III) has been found to give a higher strength in some soils. Type III cement has a finer particle size and a different compound composition than the other cement types. Chemical and physical property specifications for Portland cement can be found in ASTM C150.

The presence of organic matter and/or sulfates may have a deleterious effect on soil cement. Tests are available for detection of these materials and should be conducted if their presence is suspected.

1. *Organic matter.* A soil may be acid, neutral, or alkaline and still respond well to cement treatment. Although certain types of organic matter, such as undecomposed vegetation, may not influence stabilization adversely, organic compounds of lower molecular weight, such as nucleic acid and dextrose, act as hydration retarders and reduce strength. When such organics are present, they inhibit the normal hardening process. If the pH of a 10:1 mixture (by dry weight) of soil and cement 15 minutes after mixing is at least 12.0, it is probable that any organics present will not interfere with normal hardening.

2. *Sulfates.* Although sulfate attack is known to have an adverse effect on the quality of hardened Portland cement concrete, less is known about the sulfate resistance of cement stabilized soils. The resistance to sulfate attack differs for cement-treated, coarse-grained and fine-grained soils, and is a function of sulfate concentrations. Sulfate-clay reactions can cause deterioration of fine-grained soil-cement. On the other hand, granular soil-cements do not appear susceptible to sulfate attack. In some cases, the presence of small amounts of sulfate in the soil at the time of mixing with the cement may even be beneficial. The use of sulfate-resistant cement may not improve the resistance of clay-bearing soils, but may be effective in granular soil-cements exposed to adjacent soils and/or groundwater containing high sulfate concentrations. The use of cement for fine-grained soils containing more than about 1% sulfate should be avoided.

   For CKD, sulfates could be present in either the CKD or the soil, which could potentially react with free lime, form expansive minerals and cause swelling. The percentage of sulfates in the CKD therefore must be reported as a part of the chemical analysis provided and the soils to be stabilized should be tested for the presence of sulfate bearing minerals (often in the form of gypsum). If sulfates are present in either material, swelling tests should be conducted to evaluate the potential for the formation of expansive minerals.

6.2.2.2 Lime

The most common varieties of lime for soil stabilization are hydrated lime [$Ca(OH)_2$], quicklime [CaO], and the dolomitic variations of these high-calcium limes [$Ca(OH)_2 \cdot MgO$ and $CaO \cdot MgO$]. While hydrated lime remains the most commonly used lime stabilization admixture in the United States, use of the more caustic quicklime has grown steadily over the past two decades. Lime is usually produced by calcining limestone or dolomite, i.e., heating the limestone or dolomite to a high temperature below the melting or fusing point and thus decompose the carbonates into oxides and hydroxides. However, some lime—typically of more variable and poorer quality—is also produced as a byproduct of other chemical processes.

Typical effects of lime stabilization on the engineering properties of a variety of natural soils are shown in Figure 9-12.

Soil	Lime %	Atterberg Limits *LL*	*PL*	*PI*	Strength q_u[a]	CBR
1. CH, residual clay[b]						
(a) Site 1, Dallas–Ft.	0	63	33	30	76	
Worth Airport,	2	62	48	14	123	
residuum from Eagle	3	60	47	13	202	
Ford shale, Britton member	4	56	46	10	323	
(b) Site 2, Dallas–	0	60	27	33	70	
Ft Worth Airport,	2	48	32	16	171	
residuum from Eagle Ford	3	45	32	13	177	
shale, Tarrant member	5	48	34	14	184	
(c) Site 3, Irving, Texas,	0	76	31	45	64	
residuum from Eagle Ford	2	61	45	16	116	
shale, Britton	3	56	45	11	193	
member	5	57	45	12	302	
2. CH, Bryce silty clay,[c]	0	53	24	29	81	
Illinois, B-horizon	3	48	27	21	201	
	5	NP	NP	NP	212	
3. CH, Appling sandy loam,[d]	0	71	33	38	92	
South Carolina, residuum	3				147	
from granite	6				171	
	8				206	
4. CH, St Ann red bauxite	0	58	25	33	119	
clay loam,[d] Jamaica,	3				127	
limestone residuum	5				334	
5. CL,[e] Pelucia Creek Dam,	0	29	18	11		
Mississippi	1	32	19	13		
	2	31	22	9		
	3	30	21	9		
6. CL, Illinoian till, Illinois,[c]	0	26	15	11	43	
glacial till	3	27	21	6	126	
	5	NP	NP	NP	126	
7. SC, sandy clay, San Lorenzo,	0	54	23	31		8
Honduras[f]	5	61	38	23		20
8. MH, Surinam red earth,[d]	0	60	32	28	72	
Surinam,	3				130	
residuum from acidic	5				136	
metamorphic rock						
9. OH, organic soil with 8.1%	0	63	27	36	4	
organics[g]	2			36	4	
	4			24	8	
	8			25	7	

[a] Unconfined compressive strength in psi at 28 days unless otherwise noted; different compaction efforts used by investigators.
[b] McCallister and Petry 1990, accelerated curing.
[c] Thompson 1966.
[d] Harty 1971, 7-day cure.
[e] McElroy 1989.
[f] Personal communication with Dr. Newel Brabston, Vicksburg, Mississippi.
[g] Arman and Munfakh 1972 limits at 48 hours, q_u at 28 days, strength samples prepared with moisture content at the *LL*.

Rollings and Rollings 1996

Figure 9-12. Examples of the effects of lime stabilization on various soils.

These are the result of several chemical processes that occur after mixing the lime with the soil. Hydration of the lime absorbs water from the soil and causes an immediate drying effect. The addition of lime also introduces calcium (Ca^{+2}) and magnesium (Mg^{+2}) cations that exchange with the more active sodium (Na^{+}) and potassium (K^{+}) cations in the natural soil water chemistry; this cation exchange reduces the plasticity of the soil, which, in most cases, corresponds to a reduced swell and shrinkage potential, diminished susceptibility to strength loss with moisture, and improved workability.

The changes in the soil-water chemistry also lead to agglomeration of particles and a coarsening of the soil gradation; plastic clay soils become more like silt or sand in texture after the addition of lime. These drying, plasticity reduction, and texture effects all occur very rapidly (usually within 1 hour after the addition of lime), provided there is thorough mixing of the lime and the soil.

6.2.2.3 Fly Ash

Fly ash, also termed coal ash, is a mineral residual from the combustion of pulverized coal. Fly ash is classified according to the type of coal from which the ash was derived. Class C fly ash is derived from the burning of lignite or subbituminous coal and is often referred to as “high lime” ash because it contains a high percentage of lime. Class C fly ash is self-reactive or cementitious in the presence of water, in addition to being pozzolanic. Class F fly ash is derived from the burning of anthracite or bituminous coal and is sometimes referred to as “low lime” ash. It requires the addition of lime to form a pozzolanic reaction. To be acceptable quality, fly ash used for stabilization must meet the requirements indicated in ASTM C593 for Class C fly ash.

Fly ash contains silicon and aluminum compounds that, when mixed with lime and water, forms a hardened cementitious mass capable of obtaining high compressive strengths.

6.2.2.4 Asphalt

Asphalt stabilization is suitable for silty, sandy, and granular materials. Although asphalt (or bituminous) stabilization has been used on soils with plasticity indices of up to 15, it is more commonly used on sandy, well-graded soils with low fines contents and provides a relatively flexible platform for the pavement structure. Asphalt is used as a binder and to fill or partially fill the voids between the soil particles. It typically also provides some moisture resistance, but is not appropriate for use in poorly drained areas. The asphalt reduces the permeability of the soil by increasing the tortuosity of the pathways for water to migrate through the soil. Reduction of permeability may be relied upon to create a waterproof surface to protect underlying, water sensitive soils from the intrusion of surface water. This

mechanism must be accompanied by other aspects of the geometric design into a comprehensive system. The reduction of void spaces may also tend to alter the volume change under shear from a contractive to a dilative condition.

Much bituminous stabilization is performed in-place, with the bitumen being applied directly onto the soil or soil aggregate system, and the mixing and compaction operations being conducted immediately thereafter. For this type of construction, liquid asphalts (i.e., emulsions) are used. Emulsions are preferred over cutbacks because of energy constraints and pollution control efforts (i.e., cutbacks are no longer allowed by many agencies).

6.2.2.5 Combinations

The advantage of using combination stabilizers is that one of the stabilizers in the combination compensates for the lack of effectiveness of the other in treating a particular aspect or characteristic of a given soil. For instance, in clay areas devoid of base material, lime has been used jointly with other stabilizers, notably Portland cement or asphalt, to provide acceptable base courses. Since Portland cement or asphalt cannot be mixed successfully with plastic clays, the lime is added first to reduce the plasticity of the clay. While such stabilization practice might be more costly than the conventional single stabilizer methods, it may still prove to be economical in areas where base aggregate costs are high. Three combination stabilizers are considered in this section: lime-cement, lime-asphalt, and lime-fly ash.

1. *Lime-cement.* Lime can be used as an initial additive with Portland cement, or as the primary stabilizer. The main purpose of lime is to improve workability characteristics, mainly by reducing the plasticity of the soil. The design approach is to add enough lime to improve workability and to reduce the plasticity index to acceptable levels. The design lime content is the minimum that achieves desired results.
2. *Lime-asphalt.* Lime can be used as an initial additive with asphalt, or as the primary stabilizer. The main purpose of lime is to improve workability characteristics and to act as an anti-stripping agent for asphalt. In the latter capacity, the lime acts to neutralize acidic chemicals in the soil or aggregate that tend to interfere with bonding of the asphalt. Generally, about 1 to 2 percent lime is all that is needed for this objective. Since asphalt is the primary stabilizer, the procedures for asphalt-stabilized materials, should be followed.
3. *Lime-fly ash*. Design with lime-fly ash is somewhat different from stabilization with lime or cement. For a given combination of materials (aggregate, fly ash, and lime), a number of factors can be varied in the mix design process, such as percentage of

lime-fly ash, the moisture content, and the ratio of lime to fly ash. It is generally recognized that engineering characteristics such as strength and durability are directly related to the quality of the matrix material. The matrix material is that part consisting of fly ash, lime, and minus No. 4 aggregate fines. Basically, higher strength and improved durability are achievable when the matrix material is able to "float" the coarse aggregate particles. In effect, the fine size particles overfill the void spaces between the coarse aggregate particles. For each coarse aggregate material, there is a quantity of matrix required to effectively fill the available void spaces and to "float" the coarse aggregate particles. The quantity of matrix required for maximum dry density of the total mixture is referred to as the optimum fines content. In lime-fly ash mixtures, it is recommended that the quantity of matrix be approximately 2 percent above the optimum fines content. At the recommended fines content, the strength development is also influenced by the ratio of lime to fly ash. Adjustment of the lime-fly ash ratio will yield different values of strength and durability properties.

6.3 Design Overview

6.3.1 Design Considerations (Chemical/Binder Selection)

Design of the admixture will depend on the purpose of stabilization, i.e., reduce the moisture content, reduce the plasticity of the soil, or improve the strength. Special design considerations must also be given to the sulfate content, organic content, pH and chloride content of the soil, the presence of which could affect the performance and durability of the mix. Jones et al. (2010) provides detail design steps for addressing each of these issues.

6.3.2 Design Steps

The primary reason for stabilization is to improving the strength of the subgrade. For this application, the design of admixtures takes on a similar process regardless of the admixture type. The following design is generic to lime, cement, lime-fly ash, and lime-cement-fly ash, or asphalt admixtures:

Step 1. Classify soil to be stabilized.

(% < No. 200 sieve, % < No. 40 sieve, PI, sulfate content, etc.)

Step 2. Select appropriate admixture(s) (e.g., from Table 9-2 and Figures 9-9 through 9-12) and prepare trial mixes with varying percent content (see FHWA 2010, Army and Air Force (JDAAF) (1994) or Jones et al. (2010)). For example:

Lime: Select lowest % with pH ≈ 12.4 in 1 hour.

Cement: Use table to estimate cement content requirements.

Asphalt: Use equation and table in FHWA 2010 or Army and Air Force (JDAAF) (1994) to estimate the quantity of asphalt.

Step 3. Develop moisture-density relationship for initial design.

Step 4. Prepare triplicate samples and cure specimens at the specified density requirement (e.g., 90% AASHTO T 180 or ASTM D1557).

Use optimum water content and percent initial admixture +/- several admixture percentage points above and/or below

Step 5. Determine index strength.

Lime and Cement: Determine unconfined compressive strength (ASTM D5102 or AASHTO T 208)

Asphalt: Determine Marshall Stability (ASTM D6927 or AASHTO T 245).

Step 6. Determine resilient modulus (e.g., AASHTO T 307) for optimum percent admixture.

Perform test or estimate using correlations.

Step 7. Conduct freeze-thaw tests, regional as required for Cement, Cement-Fly Ash, Lime-Cement-Fly Ash.

Step 8. Select percent to achieve minimum design strength and freeze-thaw durability.

Step 9. Add 0.5 to 1% to compensate for non-uniform mixing.

6.3.3 Primary Design References

Details for the above design steps and specific design requirements for each type of admixture reviewed in this section are contained in the following key references.

- Army and Air Force (JDAAF). (1994). *Soil Stabilization for Pavements*. Joint Departments of the Army and Air Force, USA, TM 5-822-14/AFMAN 32-8010, Washington, D.C.
- FHWA. (2010). *Geotechnical Aspects of Pavements*. Authors: Christopher, B.R., Schwartz, C., and Boudreau, R., FHWA NHI-10-092, Federal Highway Administration, U.S. DOT, Washington, D.C., 568p.

- Jones, D., Tahim, A. Saadeh, S., and Harvey, J. (2010). *Guidelines for the Stabilization of Subgrade Soils in California*, Institute of Transportation Studies, University of California, Davis, CA.
- National Lime Association. (2004). *Lime-Treated Soil Construction Manual: Lime Stabilization & Lime Modification*, Arlington, VA.
- Portland Cement Association. (1995). *Soil-Cement Construction Handbook*. Skokie, IL.
- American Coal Ash Association. (1991). *Flexible Pavement Manual*. Washington, D.C.
- Asphalt Institute. (1979). *A Basic Emulsion Manual*, Manual Series #19, Lexington, KY.
- http://www.cement.org/index.asp and http://www.lime.org/.

6.4 Overview of Construction Specifications and Quality Assurance

6.4.1 Specification Development

Combined performance and method approach specifications are used for most chemical stabilization projects. Specifications should include acceptance criteria and minimum contractor qualifications. The specifications should state the contractor's responsibility for material selection, mix design, and quality control. Requirements for equipment, and construction methods should be identified. The specifications should also define the engineering responsibility with respect to verification of the mix design and quality assurance program to be performed during construction. Specifications should emphasize that the stabilized materials shall meet the specified resilient modulus and durability requirements for pavement support. Special constructing considerations will be required for stabilizing sulfate-rich soils and soils containing organics. The following provides an outline of general specification requirements:

- Grade correction
- Test strip
- Mixing and application equipment
- Mixing crew responsibilities
- Mixing depth and moisture content
- Mellowing before compaction (lime stabilization)
- Compaction

- Compaction quality control
- Curing
- Trafficking

With regards to trafficking, all vehicles should be kept off the stabilized layer during the specified curing period and/or until the minimum specified strength for the project has been achieved.

6.4.2 Summary of Quality Assurance

The agency should verify the mix design and conduct a quality assurance program, unless quality assurance is the responsibility of others under the contracting mechanism used. Quality assurance procedures are applied corresponding to different chemical stabilization methods at various stages of construction. It is essential to ensure that soil properties will be achieved, which provide a reliable working platform and pavement section as a result of the construction process. Critical parameters to be measured include the thickness of the stabilized layer, moisture content, stabilizer content, compaction effort and delay, curing procedures and time for the mixture, gradation of mixture, shear strength, and modulus. One of the more important quality assurance measures is the uniformity of the blend. The method to evaluate that a uniform blend is being achieved is by digging holes across the roadway and observing the mixed materials, which should have a uniform consistency, color, and moisture content. Samples should be taken for moisture content and, if possible, stabilizer additive content tests. For cement and lime stabilization, a quick assessment of uniformity can be made by spraying the sides of the hole with a phenolphthalein solution. The color of the modified soil should change to a uniform deep red indicating sufficient stabilizer in the material.

6.4.3 Summary of Instrumentation Monitoring and Construction Control

Thickness measurements and uniformity of the blend can best be made by using test holes along and across the roadway alignment. To determine the uniformity of the thickness, test pit measurements can be complimented by other thickness determinations using DCP and/or ground penetrating Radar (GPR) (ASTM D6432). Performance monitoring for pavement applications could include FWD or LFWD testing to confirm design compliance as well as obtain data for future design. Evaluation of the back calculated resilient modulus can be used to confirm that the process meets the design requirements. This value also provides a reference point for future FWD on the new pavement surface for monitoring long-term performance.

6.5 Cost Data

6.5.1 Cost Components

Typical contract pay items and units of measurement used for chemical stabilization of subgrades include the subgrade modification measured by the square yard, for all equipment, labor and incidentals necessary to mix, compact, and fine grade a chemically modified subgrade. Bid prices for subgrade modification for projects over 10,000 yd^2 have been found to range from \$2 to \$5 per cubic yard. Cement, fly ash or lime, measured by the ton for material costs with prices ranging from:

- Cement: \$100 to \$150 per ton for quantities > 150 tons
- Fly ash: \$40 to \$80 per ton for quantities > 350 tons
- Lime: \$100 to \$150 per ton for quantities > 150 tons

Most of the equipment necessary to construct a chemically modified subgrade and/or base course is common to construction activities on a highway project; therefore, additional mobilization is negligible. An exception to this could be an on-site plant used to mix a chemically modified base course (this assumes that a plant is not needed for other items of work). Mobilization costs for a plant are approximately \$25,000. Cost ranges are based on data from 2007 through 2010.

For more cost information and a conceptual cost estimating tool see *GeoTechTools*.

7.0 MOISTURE CONTROL: IMPROVED DRAINAGE - DEWATERING

7.1 Feasibility Considerations

Most soils require stabilization because they are wet and saturated. In these cases, incorporating drainage as part of subgrade stabilization can have significant construction benefits and, if the drainage is permanent, long-term pavement performance benefits. Deep drains and underdrains, typically greater than 3 feet deep, are used to remove and control the flow of groundwater. However, these types of drains can be combined with or constructed as edgedrains to also remove infiltration water that seeps into the pavement structural section. Drainable stabilization layers can be designed as drainage blanket layers to remove water from groundwater tables located close to the surface before it moves up into the pavement layers and allow infiltration water to drain vertically to the stabilization layer. The drainage blanket is either connected to the underdrains and edgedrains or daylighted at the ditch line to allow the water to exit the pavement section. The stabilization layer can also be designed as a capillary break system to intercept and remove rising capillary water and vapor movement.

In addition to providing stabilization of subgrade soils, drainage of water from the pavement system is important for the performance and service life of the pavements. The influence on design of the pavement can be significant. For example, in high rainfall areas, according to AASHTO (1993) the base section of a flexible pavement system (with a relatively thick base layer) can be reduced in thickness by as much as a factor of 2, or the design life extended by an equivalent amount, if excellent drainage is provided versus poor drainage. Likewise, an improvement in drainage leads to a reduction the slab thickness of a rigid pavement system. While the current AASHTO MEPDG (AASHTO 2008) does not directly evaluate the influence of drainage, it does assume that good to excellent drainage has been incorporated into the pavement design.

7.1.1 Applications

Consideration should be given to the use of subgrade drains whenever the following conditions exist:

- High groundwater levels, bleeding water from spring thaw, and springs that may reduce subgrade stability and provide a source of water for frost action
- Subgrade soils consisting of silts and very fine sands that may become quick or spongy when saturated
- Water seeps from underlying water-bearing strata or from subgrades in cut areas (consider intercepting drains)

- Signs of poor drainage in existing pavements such as standing water in pavement joints and ditches
- The subgrade must be stabilized to allow for rubblization of concrete pavements or for full depth reclamation of asphalt pavements.
- Wet climates

Seepage from high groundwater and springs usually can be controlled by longitudinal or transverse drains, by the use of deep foundation trench drains, or by means of graded filter blanket drains (e.g., which could be a granular stabilization layer), or combinations of these drainage methods (Cedergren 1989). For sloping terrain, interceptor trench drains can be placed outside the pavement system to intercept the lateral flow of water (e.g., from cut slopes) and remove it before it enters the pavement section. For soils that are extremely wet or saturated, consider dewatering using well points or deep horizontal drains. If drains cannot be daylighted, connection to storm drainage pipes or sump pumps may be required.

For frost susceptible soils, consider using deep horizontal drains extending below the freezing front, again either daylighted, connected to storm drainage pipes or sump pumps. Installing drains that keep the free water level at depths greater the 5 feet below the subgrade surface will usually control capillary rise (Cedergren 1989). A cut off capillary barrier consisting of a graded granular or geotextile filter/separation layer placed over the foundation soil, open graded gravel or cobbles, and a filter/separation layer placed over the aggregate layer can also be used. This layer would also provide a granular stabilization platform for construction of the pavement section. To be effective, the capillary barrier must be placed above the water table but below the freezing front and must be drained.

For existing pavements, retrofitting drainage along the edges of the pavement may also provide improvement of subgrade soil conditions without having to remove the pavement for subgrade stabilization.

Once the underdrain systems are in place and functioning, the drainage system can typically reduce subgrade pumping problems within a few days, but may take longer depending on the characteristics of the in situ materials.

7.1.2 Advantages and Potential Disadvantages

7.1.2.1 Advantages

Installing drains for subgrade stabilization has several advantages including:

- This is a well-established technology and common geotechnical practice.

- Drains are relatively fast and easy to install using readily available, relatively light weight equipment. Access onto project sites even with very poor subgrade conditions can be readily achieved.
- Can be installed prior to construction at relatively low cost leading to substantial construction savings, provided dewatering is successful.
- Enhanced performance provided by effective drainage can be used to decrease pavement design requirements and/or increase life expectancy.
- Provides a rapid renewal method for transportation facilities, minimal disruption of traffic.
- Prevents the saturation of underlying layers, speeding up construction activities.

7.1.2.2 Potential Disadvantages

There are several design disadvantages. There is also a problem in predicting the effectiveness of drains, especially in time required for improvement. Permeability and corresponding flow in soils are difficult to predict within an order of magnitude (e.g., instead of days, the improvement may take weeks). Additionally, AASHTO MEPDG (AASHTO 2008) design method does not consider the contribution of drainage in the pavement design, thus long-term cost-benefits cannot be readily evaluated.

Drainage systems rely heavily on construction for successful installation and, if contractors are not careful, drains can be compromised by construction activities. Also drains require maintenance (cleaning drains and outlets), both during and after construction to maintain their effectiveness. Finally, draining contaminated sites will likely require permits.

7.1.3 Feasibility Evaluations

7.1.3.1 Geotechnical

It is important to identify any saturated soil strata, the depth to groundwater, and subsurface water flow between soil strata. Subsurface water is especially important to recognize and identify in the transition areas between cut and fill segments. If allowed to saturate unbound base/subbase materials and subgrade soils, subsurface water can significantly decrease the strength and stiffness of these materials. Reductions in strength can result in premature surface depressions, rutting, or cracking. Seasonal moisture flow through selected soil strata can also significantly magnify the effects of differential volume change in expansive soils. Cut areas are particularly critical for subsurface water.

Soils that are subject to significant deformation and are not free draining (percent saturation exceeding 80–90%) within 2 to 4 feet of the surface are not expected to support construction traffic.

7.1.3.2 Environmental Considerations

Wet weather is a consideration for performance and may delay improvements in the subgrade, however, drainage during wet weather will likely allow construction to proceed much sooner after precipitation events than sites that do not have drainage.

Another environmental consideration is contaminated sites, which will require special permits and testing of water exiting the drainage system.

Finally, drain trenches require disposal of excavated material, which is even more of a problem with contaminated sites.

7.1.3.3 Site Conditions

Evaluate the existing conditions at the site that may indicate the need for underdrains including:

- Existing underdrains with clogged outlets on rehabilitation projects.
- Free water in the subgrade.
- Bodies of water or water sheds above and/or below the site.
- Saturated soils of moderately high permeability, such as sandy silt and silty clay or silt of low plasticity.
- Groundwater seepage through layers of permeable soil.
- Water seeping into test pits.
- Water seeping from higher elevations in cut locations.
- Water flowing on the top of the rock undercuts.
- In wet climates, if the subgrade permeability is less than 10 feet/day, some form of subsurface drainage or other design features to combat potential moisture problems should be considered.

It is important to identify sources of water, both surface and groundwater, and where the water is flowing from and to. Therefore piezometric levels should be measured along and transverse to the site. This information along with the topography is critical in determining where to collect and where to discharge water.

7.1.4 Limitations

Limitations may include:

- Large wide flat areas are difficult to drain as the exposure to surface infiltration water is relatively large, the hydraulic gradients causing inflow may be large, and the discharge areas and corresponding gradients for removal of water is small (Cedergren 1989).
- Saturated soils with more than 10% fines (minus No. 200 sieve) are not expected to be drainable with respect to supporting construction traffic unless significant time is available before construction.
- Discharge of water from contaminated sites will require special permits, and may not be allowed.

7.2 Construction and Materials

Whether drains are only used as temporary drains to dewater the subgrade for construction or the drains for subgrade stabilization will be part of the permanent pavement systems, proper construction procedures are critical for adequate performance. Care during construction to build the pavement drainage designed section without compromising the effectiveness of design is essential to the long-term pavement performance. Construction personnel (contractor and inspector) should be aware of how each construction activity can impact the performance of the pavement drainage system.

Again, significant subgrade stability improvement can be obtained by cleaning out the existing underdrain outlets on rehabilitation projects and by adding underdrains on new construction projects. If the project consists of several phases, then the Contractor should perform the outlet cleaning for the entire project at the same time to allow more time for drainage in subsequent phases.

For new construction projects, subgrade stability can be achieved by constructing the planned underdrains, or adding underdrains as soon as a water problem is found. New construction projects can allow a longer period of time for the underdrain system to work. At the beginning of construction, and certainly before winter shut down, are opportune times for this work. Construction underdrains are usually placed in the centerline of the roadway. They may also be placed in the ditch line, if the water is flowing in from a cut section at a higher elevation. The porous backfill is extended to the subgrade elevation. The outlets for the construction underdrain are made of the same pipe material and backfill as regular underdrains. The underdrains can be outlet to any convenient location. Some potential outlet locations are catch basins, manholes, pipes, or ditches. The project should not be concerned

with the contamination in the upper portion of construction underdrain backfill. Construction underdrains are sacrificial underdrains that will continue to work throughout the life of the contract and afterwards even though the upper portion is contaminated.

7.2.1 Construction

Proper grading is essential for drains to be effective. Undulating drain lines are not acceptable, as water will accumulate in depressed areas. Good practice dictates that drains must be properly connected to any drainage layers (e.g., the stabilization layer, base layers), and to outlets. Outlets are required to be set at the proper grades and ditch lines graded according to drainage requirements. Drain lines are to be carefully marked and care maintained throughout construction to avoid crushing the pipe with construction equipment (e.g., concrete trucks and other heavy vehicles/equipment are not to be allowed to travel over shallow drain lines). Permanent drains are sometimes installed after pavement construction to avoid this problem. If temporary drainage is used for stabilization, it should be maintained until the permanent drains are constructed to prevent a bathtub effect from trapping water in the pavement layers.

The filter (geotextile or aggregate) has to be carefully placed at the design location around all sides of the backfill. If open graded permeable base is used, the filter should not interfere with the flow into the drain. All drains are required to be backfilled with material at least as permeable as the permeable base.

Most states use a graded aggregate, while some states use free-draining sand. In either case, the drainage backfill should be placed below the invert of the drain pipe, and compacted to better support the drain pipe, reduce the risk of crushing the pipe, and to prevent subsequent subsidence that could affect the road. As with the trench line, the pipe must be placed at the proper grade on a level surface. Drainage backfill is placed to the final elevation and protected from fouling until the pavement section is complete. Maintaining an open drainage aggregate is critical during the remaining construction period. Construction traffic should not be allowed to traverse over the drain line. A shovel full of fines could clog the drain. The drain line could be covered with a geotextile to help prevent fouling during construction. Also, outlets must be properly drained during this phase to provide temporary drainage during construction. Ditch lines should be continuously checked and maintained, as erosion sediments could back up and foul essential features. Headwalls for outlets should be installed and outlets marked so they will not be disturbed by subsequent construction or maintenance activities.

The drain system should be inspected and tested for proper operation toward the end of construction, before final acceptance.

7.2.2 Materials

The classic drain is a trench filled with aggregate and a pipe installed near the base of the trench to facilitate rapid removal of water. The drainage aggregate in the trench drain must be both compatible with the subgrade soils with respect for filtration requirements and have adequate permeability to meet flow capacity requirements. If a stabilization layer is also to be used as a drainage blanket, the aggregate in the stabilization layer must also meet these same requirements. These diametrically opposite requirements often cannot be met by a single layer and a second filter layer (either aggregate or geotextile) may be required. For trench drains, the size of the drain will also be based on the inflow capacity of the aggregate (i.e., low permeability aggregate such as sand will be required to have a much larger trench drain perimeter than a drain constructed with open graded gravel and a geotextile filter wrap).

7.3 Design Overview

7.3.1 Design Considerations

Graded granular materials in the drain or drainage blanket require proper engineering design for the filtration compatibility requirements or they may not perform as desired. Unless flow requirements, piping resistance, clogging resistance and constructability requirements are properly specified, the soil filtration system may not properly perform. Alternatively properly designed geotextile filters can be used, if the aggregate does not meet these requirements or a more open graded aggregate with high flow potential is desirable.

Dewatering stabilization may also be used to stabilize the subgrade to allow for rubblization of concrete pavements and full depth reclamation of asphalt pavements. In some cases, the subgrade is too weak to support rubblization activities. In these cases, the potential for draining the subgrade prior to construction should be considered. Provided the drainage system can effectively remove moisture from the subgrade based on the geotechnical investigation, drains could be installed a sufficient time prior to construction to allow for adequate subgrade strength gains to permit rubblization of the concrete pavement. In this case the edgedrain should be designed to drain the subgrade as well as to collect and dispense excessive moisture immediately under the rubblized concrete and at the interface between the HMA and rubblized PCC, particularly in rolling terrain. Any existing edgedrains should be carefully inspected and repaired as necessary if they are to be left in-place.

The design of the drainage system should include pipe access installed at the "upstream" end of the drain line to gain access for camera inspection, effectiveness testing, and subsequent maintenance flushing activities. For rock or shale cuts, the design underdrains should extend

at least 6 inches into the existing rock formation. If the underdrains are too high, the water will accumulate at the rock and soil interface and cause subgrade instability.

7.3.2 Design Steps

The design steps include:

Step 1. Evaluate site conditions for the need for drainage and select the appropriate drainage alternatives for the pavement structure under consideration.

Consider deep drains and underdrains, edgedrains, interceptor drains, stabilization drainage blanket, capillary barrier, and drains for rubbilization, or combinations.

Step 2. Obtain soil samples from the site and, on the soils that are anticipated to be adjacent to the drain, perform grain size analyses to obtain D_{85}, D_{60}, D_{15}, and D_{10} and determine permeability (e.g., perform ASTM D2434). Also, specify the gradation requirement for the stabilization layer and estimate its permeability.

Step 3. Calculate anticipated flow into and through drainage system and dimension the system. Use collector pipe to reduce size of drain.

General Case

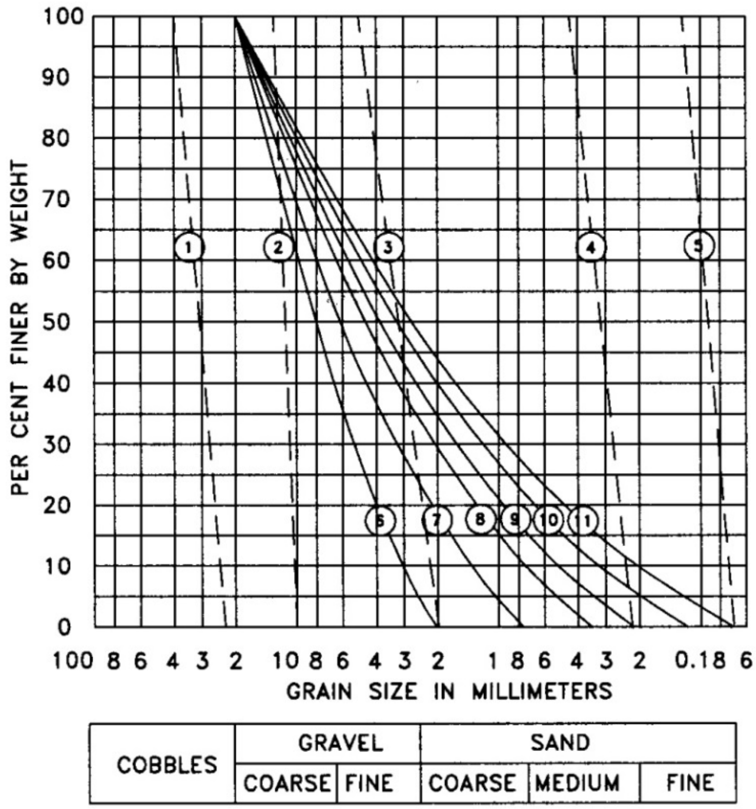
Use Darcy's Law:

$$q = k\ i\ A \qquad \text{[Eq. 9-9]}$$

where,

q	=	infiltration rate (m^3/sec)
k	=	effective permeability of soil, ft/day (m/sec)
i	=	average hydraulic gradient in soil and in drain, ft/ft (m/m)
A	=	area of soil and drain material normal to the direction of flow ft^2 (m^2)

Typical gradations and Darcy permeabilities of several aggregate and graded filter materials are shown in Figure 9-13.



COEFFICIENT OF PERMEABILITY FOR CLEAN COARSE-GRAINED DRAINAGE MATERIAL

CURVE	K, cm/sec
1	37
2	29
3	2.7
4	0.07
5	0.006
6	1.0
7	0.92
8	0.04
9	0.11
10	0.04
11	0.006

U.S. Navy 1986

Figure 9-13. Typical gradations and Darcy permeabilities of several aggregate and graded filter materials.

Use a conventional flow net analysis to calculate the hydraulic gradient (Cedergren 1989) and Darcy's Law for estimating infiltration rates into the drain; then use Darcy's Law to design the drain (i.e., calculate cross-sectional area A for flow through open-graded aggregate). Note that typical values of hydraulic gradients for roadway drainage design is less than 1 for drainage under roads, embankments, and slopes adjacent to the road, when the main source of water is precipitation.

Specific Drainage Systems

Estimates of surface infiltration, runoff infiltration rates, and drainage dimensions can be determined using accepted principles of hydraulic engineering (FHWA 1980). Specific references are as follows:

- Flow into trenches – Mansur and Kaufman 1962
- Horizontal blanket drains – Cedergren 1989
- Slope drains – Cedergren 1989

Pavement Drainage Systems

The pavement discharge rate (q_d) for sizing edgedrains and outlets to handle infiltration flow can be determined using one of the following methods

1. Pavement infiltration approach (based on estimated infiltration).
2. Permeable basic approach (based on depth-of-flow approach).
3. Time-to-drain approach (based on the time required for a specific amount of the water to drain from a saturated drainable layer, i.e. stabilization layer, subbase layer, and base layer) (e.g., see FHWA 2010).

The DRIP microcomputer program developed by FHWA can be used to rapidly evaluate the effectiveness of the drainage system and calculate the design requirements for the permeable base design, separator, and edgedrain design, including geotextile filtration requirements (http://www.me-design.com/MEDesign/DRIP.html). The program can also be used to determine the drainage path length based on pavement cross and longitudinal slopes, lane widths, edgedrain trench widths (if applicable), and cross-section geometry crowned or superelevated.

Step 4. Determine aggregate gradation to meet filtration requirements or use a geotextile designed to meet those requirements.

Piping Criterion:

$$D_{15\ Aggregate} < 5 \times D_{85\ Subgrade} \qquad \text{[Eq. 9-10]}$$

Permeability Criterion:

$$D_{15\ Aggregate} > 5 \times D_{15\ Subgrade} \qquad \text{[Eq. 9-11]}$$

Uniformity Criterion:

$$D_{50\ Aggregate} > 25 \times D_{50\ Subgrade} \qquad \text{[Eq. 9-12]}$$

Step 5. Determine outlet spacing.

The outlets must be designed to handle the flow capacity of the pipe edgedrain, which is the flow capacity of the circular pipe and can be determined from Manning's equation:

$$Q = \frac{53.01}{n} D^{8/3} S^{1/2} \qquad \text{[Eq. 9-13]}$$

where,

Q	=	Pipe capacity, ft^3/day
D	=	Pipe diameter, inches
S	=	Slope, ft/ft
n	=	Manning's roughness coefficient; 0.012 for smooth pipe and 0.024 for corrugated pipe

Once the pavement discharge rate (q_d) and the edgedrains flow capacity (Q) have been determined, the outlet spacing (L) can be sized accordingly to maintain this capacity from the following equation.

$$L \leq \frac{Q}{q_d} \quad \text{[Eq. 9-14]}$$

The maximum outlet spacing should not exceed 250 feet for maintenance purposes. A 4-inch minimum diameter smooth wall pipe at a minimum of 1% grade is also recommended.

Check ditch line elevations for the potential to daylight the stabilization drainage blanket (i.e., if part of the drainage features used).

Step 6. Prepare pavement cross sections with appropriate drainage features.

7.3.3 Primary Design References

- Cedergren, H.R. (1989). *Seepage, Drainage, and Flow Nets*. Third Edition, John Wiley and Sons, New York, NY, 465p.
- FHWA. (2010). *Geotechnical Aspects of Pavements*. Authors: Christopher, B.R., Schwartz, C., and Boudreau, R., FHWA NHI-10-092, Federal Highway Administration, U.S. DOT, Washington, D.C., 568p.
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- Mansur, C.I. and Kaufman, R.I. (1962). Dewatering, Chapter 3 in *Foundation Engineering*, G.A. Leonards, Editor, McGraw-Hill, New York, NY, pp. 241-350.
- USACE. (1992). *Engineering and Design Drainage Layers for Pavements*. Engineer Technical Letter 1110-3-435, U.S. Army Corps of Engineers, Washington, D.C.

7.4 Overview of Construction Specifications and Quality Assurance

7.4.1 Specification Development

The design and quality control of pavement drainage systems consider the following components: subgrade, subgrade stabilization/drainage blanket, and trench/edgedrain including aggregate, pipe, outlet pipe and headwall. The specifications should primarily cover the inputs and outputs relevant to the materials used for pavement drainage. Other features that should be covered include care in placement of aggregate, compaction, placement of pipe, grade of pipe, along with conformance measurements for each of these features. The specifications should require the contractor to provide a quality assurance plan which includes the conformance measurements to be provided by the contractor along with care to be taken to reduce the risk of crushing the pipe, preventing contamination of aggregate drainage layers during construction, and maintenance. Construction traffic should not be allowed to traverse over the drain line.

7.4.2 Summary of Quality Assurance

The drainage system should be inspected and tested for proper operation toward the end of construction, before final acceptance. An acceptance criterion based on performance parameters must be established, otherwise signs of poor construction practices will most likely not be identified until major structural damage is done and the pavement life has been shortened. Inspection techniques can consist of simply pouring water on the drainage layer in an upstream section of the drain and measuring the outflow against the anticipated rate. The most effective method for post-construction evaluation is video equipment (e.g., Iowa borescope and other mini-cameras). Several agencies have reported that they have improved from an edgedrain failure rate of up to 40% to a failure rate of less than 5% by improving their quality assurance program, including the use of video cameras. As a result of this type of positive experience, several states do not accept drains until video inspection indicates that they have not been damaged during construction.

7.4.3 Summary of Instrumentation Monitoring and Construction Control

Outlets should be monitored for outflow after initial installation and following precipitation events (i.e., rainfall, snow melt, ground thaw, and contractor's dust control watering). Tilt buckets or flow meters could be used to facilitate outflow monitoring and provide continuous measurements. The improvement in foundation support conditions should also be measured using DCP, FWD, and/or GPR. Probably the most significant development in edgedrain inspection has been the use of small diameter, optical tube video cameras with closed circuit

video systems. Video cameras allow the inside of the edgedrain system to be logged, and expose the weaknesses in construction and inspection procedures.

7.5 Cost Data

7.5.1 Cost Components

Typical contract pay items and units of measurement for edgedrains and underdrains are based on the underdrain by the linear foot, inclusive of the aggregate, underdrain conduit, labor, and equipment. Non-perforated outlet pipes and outlet headwalls are measured and paid for separately. The equipment used to install pavement drainage is common to highway construction projects; therefore, additional mobilization costs are negligible. The cost of a conventional gravel drain with a pipe is on the order of \$10/linear foot installed with recent bid prices ranging from \$7 to \$12/linear foot . As indicated in previously in Section 2, the cost of select granular materials ranges from \$4/yd^2 to \$10/yd^2 for a 12-inch thick granular layer. Additional cost information can be found in the *GeoTechTools* system.

8.0 MOISTURE CONTROL: IMPROVED DRAINAGE - GEOSYNTHETICS

8.1 Feasibility Considerations

Geosynthetics can be used in several ways to both enhance and directly provide drainage for pavement sections. As already discussed previously in Section 3, geosynthetic separators allow the use of open graded aggregate that can provide both improved support and drainage during the stabilization of subgrade. Geosynthetics can also be used as filters for drainage aggregate used in trench drains for interceptor drains, underdrains and edgedrains, each of which can be installed initially during construction to provide drainage. Prefabricated geocomposite drains can be used as a substitute for the aggregate in each of the types of drains.

8.1.1 Applications

There are three distinct applications of geosynthetics in drainage systems:

1. Geotextiles as filters for the edgedrains, underdrains, and free draining base.
2. Geocomposites (placed vertical) as edgedrains.
3. Geocomposites (placed horizontally beneath the base/subbase or pavement) as a horizontal base drain layer.

The first application is the most common use of geosynthetics in roadway and pavement construction. This is a well-established application, with over 40 years of successful usage. Because of their comparable performance, improved economy, consistent properties, and ease of placement, geotextiles have been used successfully to replace graded granular filters in almost all drainage applications. Geotextiles, like graded granular filters, require proper engineering design as covered in this section.

The second application, prefabricated edgedrains is also a common use of geosynthetics. However, issues of quality and poor installation practice during the early usage stifled the growth of this technology. Currently improved specifications, ASTM test standards, and installation guidelines have led to a resurgence of geocomposite usage. In addition to providing an alternate for standard aggregate filled trench type edgedrains, prefabricated geotextile edgedrains have also been used during rehabilitation projects for drainage during and post rubblization construction activities.

The last application is relatively new. Stiff, low compressible geonet composite materials with high flow capacity, originally developed for landfill applications, have been used by several state agencies to improve pavement drainage where free draining aggregates are not available or very costly. These materials have properties sufficient to handle the estimated

water flow and support traffic loads. They have been placed either below or above a dense graded base, placed as a drainage layer beneath full depth asphalt, used as a capillary break, or placed between a crack and seat concrete surface and a new asphalt overlay. When placed below the base aggregate, the geocomposite shortens the drainage path and reduces the time-to-drain. When placed directly beneath the pavement surface, the geocomposite intercepts and removes infiltration water before it enters the base and/or subgrade. The geocomposite is tied into an edgedrain system.

8.1.2 Advantages and Potential Disadvantages

8.1.2.1 Advantages

In most drainage and filtration applications, geotextile use can be justified over conventional graded granular filter material because of cost advantages including:

- The use of less-costly drainage aggregate.
- Allows the use of smaller-sized drains with the same inflow/outflow capacity.
- The possible elimination of collector pipes.
- Expedient construction, much faster than installing and compacting graded granular filters or the trench drain aggregate and pipe replaced by the geocompsite drains.
- Placement is not weather dependent.
- Lower risk of contamination and segregation of drainage aggregate during construction.
- Reduced excavation and corresponding disposal of spoil.
- Prevents the saturation of underlying layers, and facilitates the lateral drainage of water.
- Improvement of drainage performance of pavement.
- Cost effective and offers a more sustainability system than use of gravel.

In addition, geosynthetics often increase drainage system reliability and, considering the value of drainage in geotechnical engineering, a significant cost-benefit can result when the designer is assured of a properly performing drain**.**

8.2.1.2 Potential Disadvantages

Potential disadvantages include:

- Relatively (compared to drain stone) easy to damage requiring greater care during construction.
- Geocomposite edgedrains used in pavements can be difficult to maintain.
- Limited demonstration of life-cycle cost benefits for horizontal geocomposite drain application. In projects using recycled concrete, rubblizing, or crack-and-seat techniques, geotextiles and granular filters are susceptible to clogging by precipitants.

8.1.3 Feasibility Evaluations

8.1.3.1 Geotechnical

The same considerations, which were covered in Section 7.1.3.1 for determining drainage requirements, also apply to geosynthetic drainage systems. The key geotechnical components will be to identify the soils to be drained and the hydraulic characteristics of those soils for determining the filtration properties of the geosynthetic.

8.1.3.2 Environmental Considerations

Except for easier wet weather construction than granular drainage layers, the same environmental considerations apply to geosynthetic drainage systems. There will be less spoil to dispose of when using geosynthetic filters or geocomposite drains as smaller trenches can be used to maintain the same flow capacity; however, some spoil will still be produced and requires disposal. Information on the pH of the groundwater and alkaline or acidic conditions is required for evaluating geosynthetic durability and selecting the most durable polymers for the site conditions.

8.1.3.3 Site Conditions

Again, evaluation of site conditions should consider the same items as in Section 7.1.3.3. In addition, the availability of on-site aggregates should be explored as this may be a more cost effective option to using geosynthetics. The pH of the groundwater and any previous use of lime stabilization may exclude the use of geosynthetics manufactured with polyester polymers.

8.1.4 Limitations

Geotextiles (and granular filters) are susceptible to clogging by tufa precipitate from RCM and should not be indiscriminately used to separate RCM from the drain or wrapped around

pipes. Environmental conditions noted in Section 8.1.3.2 may limit the geosynthetics to be considered for the project.

Geotextiles should not be placed between the RCM and the drain, but could be placed beneath and on the outside of the drain to prevent infiltration of the subgrade and subbase layers.

8.2 Construction and Materials

8.2.1 Construction

When placing a geotextile filter for any application, the geotextile must be placed such that it is in intimate contact with the soil to be filtered. There should be no void space between the geotextile and soil. The geotextile filter will be used to line the edgedrain trench to prevent migration of fines from the surrounding soil into the drainage trench. However, the top of the trench adjacent to the permeable base should be left open to allow a direct path for water into the drainage pipe.

For all drainage applications, the following construction steps should be followed:

Step 1. The surface on which the geotextile is to be placed should be excavated to design grade to provide a smooth, graded surface free of debris and large cavities.

Step 2. Between preparation of the subgrade and construction of the system, the geotextile should be well protected to prevent any degradation due to exposure to the elements.

Step 3. After excavating to design grade, the geotextile should be cut (if required) to the desired width (including allowances for non-tight placement in trenches and overlaps of the ends of adjacent rolls) or cut after placement of the drainage aggregate in the trench.

Step 4. Care should be taken during construction to avoid contamination of the geotextile. If it becomes contaminated, it must be removed and replaced with new material.

Step 5. In drainage systems, the geotextile should be placed with the machine direction (i.e., in line with the roll) following the direction of water flow; for pavements, the geotextile should be parallel to the roadway. It should be placed loosely (not taut), but without wrinkles or folds. Care should be taken to place the geotextile in intimate contact with the soil so that no void spaces occur behind it.

Step 6. The ends for subsequent rolls and parallel rolls of geotextile should be overlapped a minimum of 1 foot in roadways and 1 to 2 feet in drains, depending on the anticipated severity of hydraulic flow and the placement conditions. For high hydraulic flow conditions

and heavy construction, such as with deep trenches or large stones, the overlaps should be increased. For large open sites using base drains, overlaps should be pinned or anchored to hold the geotextile in place until placement of the aggregate. The upstream geotextile should always overlap over the downstream geotextile.

Step 7. To limit exposure of the geotextile to sunlight, dirt, damage, etc., placement of drainage or roadway base aggregate should proceed immediately following placement of the geotextile. The geotextile should be covered with a minimum of 1 foot of loosely placed aggregate prior to compaction. If thinner lifts are used, higher survivability fabrics may be required. For drainage trenches, at least 4 inches of drainage stone should be placed as a bedding layer below the slotted collector pipe (if required), with additional aggregate placed to the minimum required construction depth. Compaction is necessary to seat the drainage system against the natural soil and to reduce settlement within the drain. The aggregate should be compacted with vibratory equipment to a minimum of 95% Standard AASHTO T 99 density unless the trench is required for structural support. If higher compactive efforts are required, the geotextiles meeting the survivability property values listed in Table 9-7 should be utilized.

Step 8. After compaction, for trench drains, the two protruding edges of the geotextile should be overlapped at the top of the compacted granular drainage material. A minimum overlap of 1 foot is recommended to ensure complete coverage of the trench width. The overlap is important because it protects the drainage aggregate from surface contamination. After completing the overlap, backfill should be placed and compacted to the desired final grade.

A schematic of the construction procedures for a geotextile-lined underdrain trench is shown in Figure 9-14.

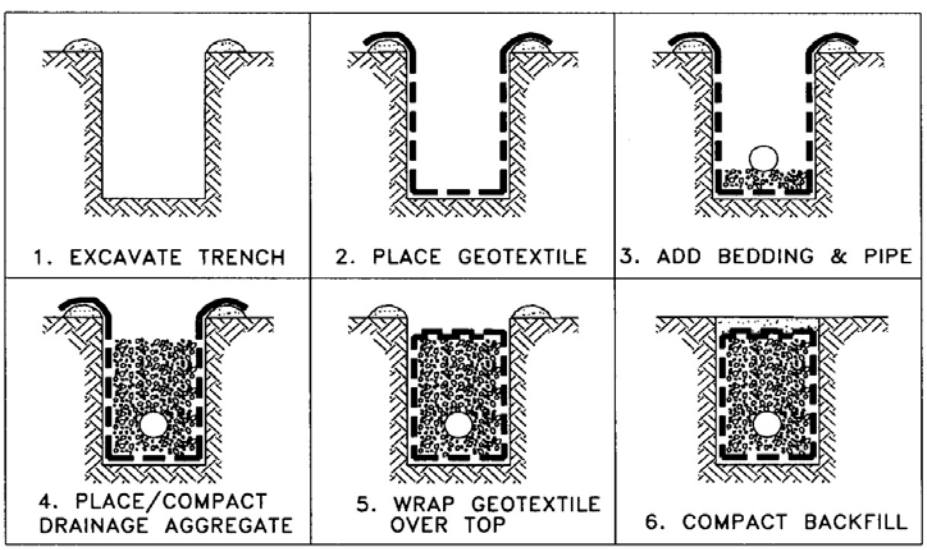


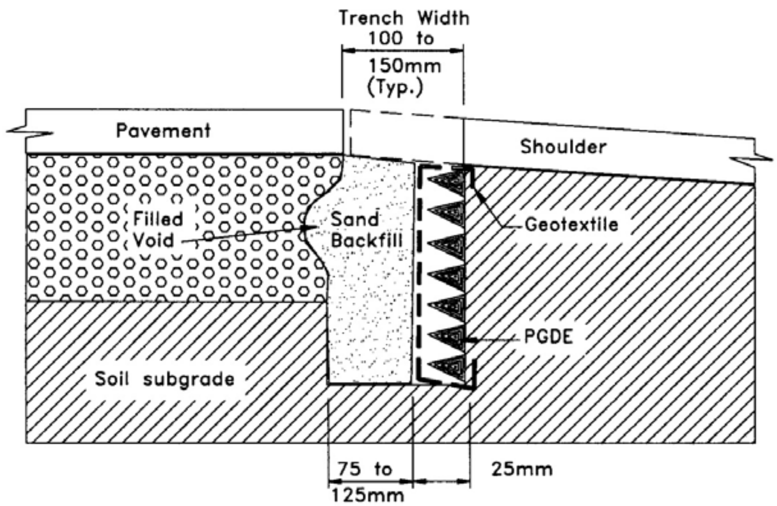
Figure 9-14. Construction procedure for geotextile-lined underdrains.

A geocomposite, or prefabricated, drain consists of a geotextile filter and a water collection and conveyance core. The cores convey the water and are generally made of plastic waffles, three-dimensional meshes or mats, extruded and fluted plastic sheets, or nets. A wide variety of geocomposites are readily available. For edgedrain design, only geocomposites that allow two-sided flow (i.e., flow into the drains from both sides) should be used.

Key installation issues affecting the performance of edgedrain systems, such as maintaining the verticality of the drain panel (geotextile or geocomposite) in the trench, proper positioning of the drain panel within the trench, backfilling with open graded aggregate, timely installation of outlet fittings and pipes, and the use of outlet pipes with adequate pipe stiffness, should be taken into account. The following are considerations specific to the installation of geocomposite drains:

1. As with all geosynthetic applications, care should be taken during storage and placement to avoid damage to the material.
2. Placement of the backfill directly against the geotextile filter must be closely observed, and compaction of soil with equipment directly against the geocomposite should be avoided. Otherwise, the filter could be damaged or the drain could even be crushed. Use of clean granular backfill reduces the compaction energy requirements.
3. At the joints, where the sheets or strips of geocomposite butt together, the geotextile filter must be carefully overlapped to prevent soil infiltration. Also, the geotextile should extend beyond the ends of the drain to prevent soil from entering at the edges.
4. Details must be provided on how the prefabricated drains tie into the collector drainage systems.

The recommended detail for proper edgedrain installation is shown in Figure 9-15.



Koerner et al. 1994

Figure 9-15. Recommended installation method for prefabricated geocomposite edgedrains.

Note that sand backfill is placed between the aggregate and the geocomposite in order to maintain intimate contact between the base layer and the geotextile. Additional recommendations can be found in ASTM D6088 *Practice for Installation of Geocomposite Edgedrains*.

For installation of the horizontal geocomposite drain, the geocomposite material is placed in such a way that its roll width is perpendicular to the roadway centerline on the prepared subgrade. Aggregate subbase is end dumped onto the geocomposite and spread with a bull-dozer and compacted with a smooth-drum vibratory roller. Compaction of the first lift above the geocomposite is done with a dozer initially and then with a smooth-drum roller with the vibrator turned off.

8.2.2 Geosynthetics

8.2.2.1 Geotextile Filters

As with conventional aggregate drainage systems, geotextile filter design and selection should be based on the grain size of the material to be protected, permeability requirements, clogging resistance, and physical property requirements. The key properties will include the geotextile permittivity, opening size as defined by the Apparent Opening Size (AOS) and, in order to provide support for construction traffic, the geotextile must also satisfy survivability and endurance criteria.

In projects using RCM or where concrete pavements will be rubbilized, geotextiles and granular filters are susceptible to clogging by precipitate and should not be indiscriminately

used to separate the permeable base from the drain or wrapped around pipes. Geotextiles should not be placed between the recycled material and the drain, but could be placed beneath and on the outside of the drain to prevent infiltration of the subgrade and subbase layers.

8.2.2.2 Edgedrains

The geocomposite drain property requirements are related to design considerations including:

- Geotextile filtration/clogging.
- Long-term compressive strength of polymeric core.
- Reduction of flow capacity due to intrusion of geotextile into the core.
- Long-term inflow/outflow capacity.

The geotextiles filter used on the edgedrains must be evaluated for meeting the filtration requirements indicated in Section 8.2.2.1 in this chapter. If the geotextile supplied with the geocomposite is not appropriate for the design conditions, system safety will be compromised. Other important properties of the geocomposite are the in-plane flow capacity of the geocomposite edgedrain. The maximum seepage flow into the system must be estimated using the procedure described in Section 7 of this Chapter. Then the geocomposite is selected on the basis of these seepage requirements. The geocomposite must also be sufficiently strong to survive the installation and compaction of gravel around the geocomposite and must meet short-term and long-term strength requirements. Long-term design pressure on a geocomposite core should be limited to either of the following (FHWA 1993):

- The maximum pressure sustained on the core in a test of 10,000 hours minimum duration; or
- The crushing pressure of a core, as defined with a quick loading test, divided by a safety factor of five.

Installation details, such as joining adjacent sections of the geocomposite and connections to outlets, are usually product specific. Product specific variances should be considered and addressed in the design, specification, detailing and construction phases of a project. Finally consideration should be given to the drainage system performance factors such as distance between drain outlets, hydraulic gradient of the drains, potential for blockage due to small animals, freezing, etc.

8.2.2.3 Horizontal Geocomposite Drains

The geocomposite must have the stiffness required to support traffic without significant deformation under cyclic traffic loading. At the same time, the geocomposite must have a flow capacity to rapidly drain the pavement section and prevent saturation of the base. Outflow capacity in relation to the requirements for a roadway system typically require complete drainage within 2 hours. Conventionally, a 4-inch thick open-graded base layer has proven adequate to meet the flow requirement (Christopher and McGuffey 1997). This layer has a minimum permeability of 1000 ft/day and preferably 2000 to 3000 ft/day. For comparison with a geocomposite, this layer would have a transmissivity of 0.004 to 0.01 ft^3/sec . With a typical roadway gradient of 0.02 (for a 2% grade), this layer provides a flow capacity ranging from 6 to 22 ft^3/day/ft length of road.

8.3 Design Overview

8.3.1 Design Considerations

As previously indicated in Section 8.1, there are three distinct applications of geosynthetics in drainage systems, each of which requires similar design for the geotextile filter; however unique design requirements for the composite applications:

1. Geotextiles as filters for the edgedrains, underdrains, and free draining base
2. Geocomposites (placed vertically) as edgedrains
3. Geocomposites (placed horizontally beneath the base/subbase or pavement) as a horizontal base drain layer

The following design steps for each of the applications are from the FHWA *Geosynthetic Design and Construction Guidelines Manual* (FHWA 2008). The level of design and testing required depends on the critical nature of the project and the severity of the hydraulic and soil conditions. Especially for critical projects, consideration of the risks and the consequences of geotextile filter failure require great care in selecting the appropriate geotextile. For such projects, and for severe hydraulic conditions, very conservative designs are recommended. **Geotextile selection should not be based on cost alone.** The cost of the geotextile is usually minor in comparison to the other components and the construction costs of a drainage system. Also, do not try to save money by eliminating laboratory soil-geotextile performance testing when such testing is required by the design procedure.

A National Cooperative Highway Research Program (NCHRP) study by Koerner et al. (1994) on the performance of geotextiles in drainage systems indicated that the FHWA

design criteria were an excellent predictor of filter performance, particularly for granular soils (<50% passing a No. 200 sieve).

8.3.2 Design Steps

8.3.2.1 Filtration Design for Geotextile Filters

The geotextile filtration characteristics must be checked for compatibility with the gradation and permeability of the subgrade. The requirements for proper performance can be appropriately selected by using the following design steps:

Step 1. Determine the gradation of the material to be filtered. The filtered material is the soil directly adjacent to the drain. Determine D_{85}, D_{15} and percent finer than a No. 200 sieve.

Step 2. Determine the permeability of the soil located directly adjacent to the geotextile filter.

Step 3. Apply design criteria to determine apparent open size (AOS), permeability (k), and permittivity (ψ) requirements for the geotextile as initially reviewed in Section 3.3.2 (after FHWA 2008):

$$AOS \leq D_{85\ Subgrade}\ (Wovens)$$ [Eq. 9-6]

$$AOS \leq 1.8 \times D_{85\ Subgrade}\ (Nonwovens)$$ [Eq. 9-7]

$$k_{Geotextile} > k_{Soil}\ and\ \psi \geq 0.1\sec^{-1}$$ [Eq. 9-8]

When using Equation 9-7 for noncohesive silts and other highly pumping susceptible soils, a filter bridge may not develop, especially under dynamic, pulsating flow. A conservative (smaller) AOS ≤ $D_{85subgrade}$ is advised, and laboratory filtration tests are recommended.

For filter applications, the permittivity requirements (ASTM D4491) are greater than for separation shown in Section 3 and depend on the percentage of fines in the soil to be filtered. The more fines in the soil, the greater the permittivity required. The following equations are recommended based on satisfactory past performance of geotextile filters.

For > 50% passing No. 200 (0.075 mm)

$$\psi \geq 0.1\sec^{-1}$$ [Eq. 9-8a]

For 15 to 50% passing No. 200 (0.075 mm)

$\psi \geq 0.2 \sec^{-1}$ [Eq. 9-8b]

For < 15% passing No. 200 (0.075 mm)

$\psi \geq 0.5 \sec^{-1}$ [Eq. 9-8c]

For filter applications, a clogging criterion should also be applied:

1. For steady state flow, low hydraulic gradient and well graded or uniform upstream soil, the clogging criterion is:

   $AOS \geq 3 \times D_{15\,Upstream\ Soil}$ [Eq. 9-15]

   This equation applies to soils with $C_u > 3$. For soils with $C_u \leq 3$, a geotextile with the maximum AOS value from the retention criterion should be used.

2. Other qualifiers:
   - Nonwoven geotextiles: Porosity (geotextile) $\geq$ 50%
   - Woven geotextiles: Percent open area $\geq$ 4%

3. **Alternative:** Run filtration tests, especially for critical and severe applications.

Step 4. Determine the geotextile survivability requirement. In order to perform effectively, the geotextile must also survive the installation process. AASHTO M 288 (2014a) provides the criteria for geotextile strength required to survive construction of roads, as shown in Table 9-10.

Table 9-10. Geotextile Survivability Requirements

Property	ASTM Test Method	Units	Geotextile Class 1 and Elongation < 50%	Geotextile Class 1 and Elongation ≥ 50%	Geotextile Class 2 and Elongation < 50%	Geotextile Class 2 and Elongation ≥ 50%
Grab Strength	D4632	lb	315	200	250	157
Seam Strength	D4632	lb	260	180	220	140
Tear Strength	D4533	lb	110	80	90	56
Puncture Strength	D6241	lb	620	433	495	309
Ultraviolet Stability (Retained Strength)	D4355	%	50% after 500 hours of exposure	50% after 500 hours of exposure	50% after 500 hours of exposure	50% after 500 hours of exposure

Source: AASHTO 2014a

Note: Elongation measured in accordance with ASTM D4632 with < 50% typical of woven geotextiles and > 50% typical of nonwoven geotextiles.

Use Class 2 where a moderate level of survivability is required where filters are used in edgedrains and for separation layers where subgrade CBR > 3 and normal weight construction equipment is anticipated. For separation layers, a minimum of 6 inches of base/subbase should be maintained between the wheel and geotextile at all times. For geotextile filters used in drainage blankets and capillary barriers, use Class 1 geotextiles for CBR < 3 and when heavy construction equipment is anticipated.

8.3.2.2 Prefabricated Geocomposite Drain Design

For the design and selection of geocomposite drainage systems, either a prefabricated geocomposite edgedrain (PGED) or a horizontal geocomposite drainage blanket, the following three basic design steps should be followed:

Step 1. Design for adequate filtration without clogging or piping.

See Section 8.3.2.1 and determine the apparent open size (AOS), permeability (k), and permittivity (ψ) requirements for the geotextile.

Step 2. Design for adequate inflow/outflow capacity under design loads to provide maximum anticipated seepage during design life.

In order to design the in-plane flow capacity of a geotextile or the flow capacity of the core or a geocomposite, the maximum seepage flow into the system (q_d) must be estimated using the procedure described in Section 7.3.2, Step 3. (Note: for evaluating the horizontal drainage blanket, the inflow gradient is vertical (i = 1) and the outflow is based gradient for the slope of the drain (i = S_L).) Then the geocomposite is selected on the basis of these seepage requirements. The flow capacity of the geocomposite or geotextile can be determined from the transmissivity of the material. The test for transmissivity is ASTM D4716 *Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products*. The maximum flow capacity per unit width of the geocomposite (i.e., a geocomposite horizontal drain) can then be calculated using Darcy's Law:

$$q = k_p\, i\, A = k_p\, i\, B\, t \qquad \text{[Eq. 9-16]}$$

or

$$q/B = \theta\, i \qquad \text{[Eq. 9-17]}$$

where:

q	=	flow rate (L^3/T)
k_p	=	in-plane coefficient of permeability for the geosynthetic (L/T)
B	=	width of geosynthetic (L)
t	=	thickness of geosynthetic (L)
θ	=	transmissivity of geosynthetic (= k_pt) (L^2/T)
i	=	hydraulic gradient (L/L)

The flow capacity of a vertically placed geocomposite edgedrain is given by the following equation:

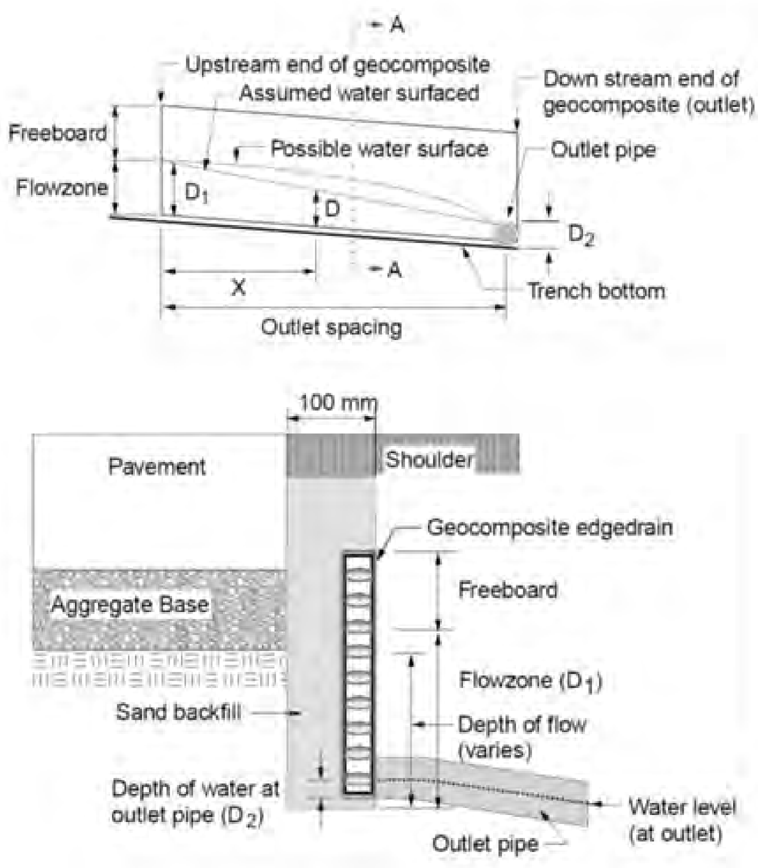
$$Q = C\,D\left(S + \frac{D_1 - D_2}{L}\right)^{1/2} \qquad \text{[Eq. 9-18]}$$

where:

Q	=	Geocomposite edgedrain capacity, $ft^3/day/ft$

C	=	Manufacturer supplied prefabricated edgedrain (PGED) flow factor, ft^3/day/inch (typical range = 0.5 to 2.5)
D	=	Averaged depth of flow = $(D_1 + D_2)/2$, inches
D_1	=	Depth of flow zone, ft
D_2	=	Depth of outlet (outlet pipe diameter), ft
S	=	Longitudinal slope, ft/ft
L	=	Outlet spacing, ft

A schematic diagram of flow in a prefabricated edgedrain (PGED), illustrating the various inputs to the equation above is shown in Figure 9-16.



FHWA 1998

Figure 9-16. Schematic diagrams for computing flow of PGED: a roadway cross section (top) and an edgedain cross section (bottom).

8.3.3 Primary Design References

- FHWA. (2010). *Geotechnical Aspects of Pavements*. Authors: Christopher, B.R., Schwartz, C., and Boudreau, R., FHWA NHI-10-092, Federal Highway Administration, U.S. DOT, Washington, D.C., 568p.
- FHWA. (1999). *Pavement Subsurface Drainage Design*. Authors: ERES Consultants, Inc., Participants Reference Manual for NHI Course Number 131026, National Highway Institute, Federal Highway Administration, U.S. DOT, Arlington, VA.
- FHWA. (2008). *Geosynthetic Design and Construction Guidelines*. Authors: Holtz, R.D., Christopher, B.R., and Berg, R.R., FHWA-HI-07-092, Federal Highway Administration, U.S. DOT, Washington, D.C., 460p.
- NCHRP 1-37A (2004). *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, Final Report*. Authors: ARA, Inc., ERES Consultants Division, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, D.C.

8.4 Geosynthetics Overview of Construction Specifications and Quality Assurance

8.4.1 Specification Development

Guide geotextile specifications for routine drainage and filtration applications and use in trench drain systems are included in AASHTO M 288 (2014a) standard specifications for geotextiles. The actual hydraulic and physical properties of the geotextile must be selected by considering the nature of the project (critical/less critical), hydraulic conditions (severe/less severe), soil conditions at the site, and construction and installation procedures appropriate for the project. ASTM has developed a standard specification for *Geocomposites for Pavement Edgedrains and Other High-Flow Applications* (ASTM D7001).

8.4.2 Summary of Quality Assurance

Special attention should be given to aggregate placement and potential for geotextile damage. Also, maintaining the appropriate geotextile overlap at the top of the trench and at roll ends is especially important. The following provides an installation check list for geotextile filters used in trench drains:

1. Check the excavation of the drain trench along the shoulder making sure that there are no large voids or sharp protrusions along the sides or bottom.

2. Observe the geotextile being rolled out over the trench. Make sure that soil excavated from the trench or loose soil along the sides of the trench does not fall into the trench or on top of the geotextile.
3. After placement, check to make sure that there are no wrinkles or folds in the geotextile and the geotextile is placed flat against (in complete contact with) the soil along the bottom, sides and in the corners.
4. Observe initial placement of aggregate (typically ¾-inch gravel) over the geotextile in the bottom of the trench and compaction of the aggregate to the thickness indicated on the drawings before placing the pipe. (This is a bedding layer for the pipe.)
5. In one small section of the trench, uncover the geotextile, removing about 1 square foot of stone, and check for damage. If damaged, the contractor must change the placement procedure by reducing the drop height onto the geotextile.
6. Monitor the placement of the pipe and the stone around and on top of the pipe. Confirm that the stone is choked under the edges of the pipe and that the stone is compacted in accordance with the specification requirements.
7. Check that the geotextile is folded over the top of the gravel in the trench (i.e., the drainage media) and that the overlap is a minimum of 1 foot or the width of the trench if smaller than 1 foot.
8. Confirm that the overlap is maintained during placement of cover soil or asphalt over the geotextile.

8.4.3 Summary of Instrumentation Monitoring and Construction Control

As with conventional aggregate drains, in order to evaluate the performance and effectiveness of geotextile filters and geocomposite drains, the drain outlets should be monitored for outflow after initial installation and following precipitation events (i.e., rainfall, snow melt, ground thaw, and contractors dust control watering). Tilt buckets or flow meters could be used to facilitate outflow monitoring and provide continuous measurements. The improvement in foundation support conditions should also be measured using DCP, FWD and/or GPR. Video inspection of all outlets and drain lines should be required on all projects.

8.5 Cost Data

8.5.1 Cost Components

In general, the cost of the geotextile material in drainage applications will typically range from \$1.00 to \$1.50/yd^2, depending upon the type specified and quantity ordered. Installation

costs will depend upon the project difficulty and contractor's experience; typically, they range from \$0.50 to \$1.50/yd^2 of geotextile. Higher costs should be anticipated for below-water placement. Labor installation costs for the geotextile are easily repaid because construction can proceed at a faster pace, less care is needed to prevent segregation and contamination of granular filter materials, and multilayered granular filters are typically not necessary.

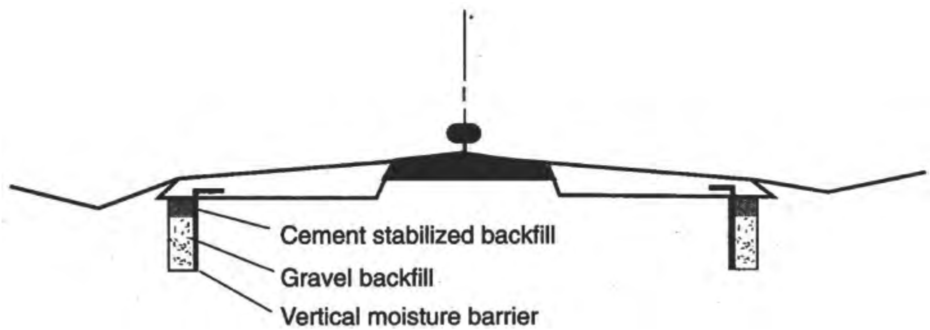
Cost of prefabricated drains typically ranges from \$0.75 to \$1.00/yd^2. The high material cost is usually offset by expedient construction and reduction in required quantities of select granular materials. For example, geocomposites used for pavement edgedrains typically cost \$1.00 to \$3.00/linear foot installed while a conventional geotextile wrapped gravel drain with a pipe is on the order of \$9.00/linear foot installed. Additional cost information on geosynthetics in drainage applications is available on the *GeoTechTools* website.

9.0 MOISTURE CONTROL: PARTIAL AND FULL ENCAPSULATION

9.1 Feasibility Considerations

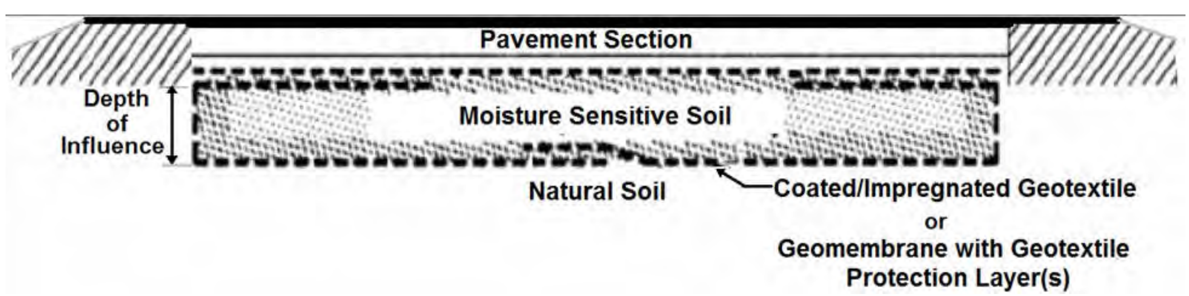
9.1.1 Applications

When expansive soils are encountered along a project in environments and areas where significant moisture fluctuations in the subgrade are expected, partial encapsulation along the edge of the pavement (see Figure 9-17) or full encapsulation of the expansive soil (see Figure 9-18) can be used to reduce change in subgrade moisture and retard swelling and/or shrinkage of expansive soils.



Wyoming DOT

Figure 9-17. Partial encapsulation cross section.



After Koerner and Hwu 1989

Figure 9-18. Full encapsulation cross section.

Full encapsulation refers to completely surrounding the soil with a geomembranes to prevent moisture from entering or exiting the expansive soil. Partial encapsulation is used to minimize water from entering or exiting through expansive soils from pavement surface, joints, cracks, and edges of the roadway subgrade.

Partial encapsulation can be used to minimize the post construction problems encountered in expansive soils. The geomembrane is used as a moisture barrier and is placed in horizontal, vertical, or in both directions. Preventing moisture from entering or leaving expansive soils minimizes or prevents pavement distresses, formation of cracks, and slab distortion. This

method provides an alternate that delays potential problems with expansive soils. It does not completely eliminate the problem; however, the only permanent solution is to remove and replace the expansive soil with sufficient overburden to resist the swell potential, which is not always feasible.

9.1.2 Advantages and Potential Disadvantages

9.2.1.1 Advantages

- Relatively fast and easy placement.
- Prevents/minimizes the damage that would be caused by long-term moisture changes.
- Suitable for rapid renewal of transportation facilities.
- Provides long-lived facilities.

9.1.2.2 Potential Disadvantages

- Lack of simple, comprehensive, reliable, and non-proprietary analysis and design procedures.
- Lack of established engineering parameters and/or performance criteria, especially with respect to long-term performance predictions.
- Lack of easy-to-use tools for technology selection.

9.1.3 Feasibility Evaluations

9.1.3.1 Geotechnical

Swelling or expansive soils are susceptible to volume change (shrink and swell) with seasonal fluctuations in moisture content. The magnitude of this volume change is dependent on the type of soil (shrink-swell potential) and its change in moisture content. A loss of moisture will cause the soil to shrink, while an increase in moisture will cause it to expand or swell. This volume change of clay type soils can result in longitudinal cracks near the pavement's edge and significant surface roughness (varying swells and depressions) along the pavement's length.

The key geotechnical issue will be determining the depth of soil requiring encapsulation or the depth of moisture barrier penetration for partial encapsulation. The depth required for full encapsulation will depend on the existing moisture content of the soil below the barrier, the affinity for absorbing additional moisture, the swell pressure and correspondingly the overburden requirements to prevent additional swell. For partial encapsulation, the depth will depend on the criteria for the allowable moisture change within the encapsulated soil for a

specified time in relation to the diffusion coefficient of the membrane and the active zone of moisture activity.

9.1.3.2 Environmental Considerations

Environmental factors such as temperature, temperature variation, humidity, rainfall, and wind must be considered. Geosynthetic liners require special consideration in their selection and installation techniques for cold weather construction and should not be installed during rain events. Some geosynthetic barriers are more sensitive to temperature than others, and moisture and wind affect field-seaming ability. Barriers constructed of field impregnated coated geotextiles must be placed during carefully defined weather conditions.

9.1.3.3 Site Conditions

Site conditions will include the type of soil, in situ moisture content and plasticity of the soil, and the thickness and depth of expansive soil layers. The stiffness of the subgrade materials is also required to determine if additional improvement is necessary. It will also be important to locate the long term groundwater level as well as any perched water level(s) and corresponding seepage flow conditions at the site (i.e., where the water is flowing from) in order to evaluate the need for interceptor drains to limit water access to the expansive materials. Identification of rocky soil layers will be important to determine the protection requirements at the base of excavations and for such materials that may be used as fill over the liner.

9.1.4 Limitations

As indicated above, it may not be feasible to place liners in inclement weather conditions. Also, partial encapsulation must consider long-term maintenance in the selection of this technology.

9.2 Construction and Materials

9.2.1 Construction

The subgrade material, subgrade preparation, panel deployment method, overlying soil fill type, and placement and compaction of overlying fill soil all affect the geosynthetic barrier's survivability.

The key construction activities for *full encapsulation* include:

- Excavation of the entire soil mass to the appropriate depth.

- Preparing a level surface.
- Spraying the base of the excavation with bitumen at approximately 1 gal/yd^2, which provides better resistance to slippage, additional liner protection from puncture and an extra sealing layer.
- Placement of the geomembrane or the geotextile which will be coated with bitumen after placement on the base of the excavation and extending up and over the side slopes at a sufficient distance to allow for complete encapsulation of the surface.
- Applying bitumen to the geotextile, if used for the liner.
- Backfilling and compaction in appropriate lifts to the required grade.
- Folding over the geosynthetic liner, overlapping and sealing the seams as required.
- Placement of a roadbed layer over the surface of the liner.

For *partial encapsulation*, the construction steps include:

- Excavation of a trench on both sides of the roadway in the area to be encapsulated.
- The width and depth of the trench vary from 2.5 to 4 inches and 5 to 10 feet, respectively.
- The geomembrane is laid into a trench.
    - The geomembrane is placed on the shoulder side of the trench rather than the pavement side.
    - A specially designed dispenser is used, which is carried horizontally and inserted vertically into the trench by passing the geomembranes around a bar at a 45 degree angle.
    - For coated geotextiles, the coated side is often placed against the trench wall to avoid any damage from backfilling operation.
    - A tougher, thicker geomembrane should be used if damage from construction.
- Overlaps are seamed as required (i.e., welding geomembranes or spraying bitumen between geotextile overlaps for coated geotextiles).
- Flowable fill, consisting of medium-graded sand, a small percentage of cement, and a large portion of high fly ash and water, is used to backfill the trench so that the geomembrane will not be damaged.
- The geomembranes from both sides are then extended over the subgrade surface for the required distance, ideally over the entire surface between the trench lines and seamed at the centerline of the roadway.

9.2.2 Materials

Most standard geomembranes can be used including Chlorosulfonated Polyethylene (CSPE), Ethylene Interpolymer Alloy (EIA), Ethylene Propylene Diene Terpolymer (EPDM), High-Density Polyethylene (HDPE), Linear Low-Density Polyethylene (LLDPE), Polypropylene (PP), or Polyvinyl Chloride (PVC) as well as others. The geomembranes are supplied in roll form with lengths varying from 650 to 1,000 feet and widths of approximately 15 to 35 feet or may be delivered in panels of a specific size. It is important to order widths that are appropriate for the design dimensions to minimize field cutting and seaming. Typical membrane thickness for encapsulation applications are 40 mils. A nonwoven geotextile cushion layer will be required above the liner to protect it during placement of fill over the liner. A roughened sheet geomembrane should be considered to avoid slippage of the geotextile and consequently the roadbed material over the surface of the geomembrane. For full encapsulation, if rocky soils are encountered at the base of the excavation, a geotextile cushion layer will also be required beneath the liner to protect it from puncture during installation as well as to provide enhanced long-term puncture resistance.

As previously indicated, the liner can also be constructed in-place using impregnated geotextiles. The coating treatment is applied in the field, after the geotextile is deployed. A nonwoven geotextile is used with a variety of coatings, including asphalt, rubber-bitumen, emulsified asphalt, or polymeric formulations. The coating may be proprietary. The geotextile type and mass per unit area will be a function of the coating treatment, although use of lightweight nonwoven geotextiles, in the range of 6 to 12 oz/yd^2, is common. The barrier is formed as sprayed-on liquid solidifies into a seam-free membrane. Although sprayed-on membranes are seam-free, bubbles and pinholes may form during installation and can cause performance problems. Proper preparation of the geotextile (i.e., clean and dry) to be sprayed is important.

As indicated above, a nonwoven geotextile will be required for rocky subgrade soils below a geomembrane barrier to enhance its puncture resistance during installation and in-service. Geotextiles act as cushions and further prevent puncture of the geomembrane. For this application, 8 oz/yd^2 needled punched nonwoven geotextiles are typically used as the protection layer.

9.3 Design Overview

9.3.1 Design Considerations

Current design methods for the geomembrane are based primarily on empirical evidence and rational principles. The main design issue is to determine the minimum depth of installation. Two design methods are available:

- Determining the depth of vertical moisture barrier based on the climatic conditions (Picornell-Darder 1985), and
- Determining the water diffusion through membrane based on the diffusion coefficient of soil (Lord and Koerner 1986).

Another consideration should be the depth to provide a stable condition (i.e., no moisture change) such that the overburden pressure from the thickness of the encapsulated material will prevent heave based on swell pressure test results.

9.3.2 Design Steps

The design steps include:

Step 1. Determine the depth to the base of the encapsulation layer or the vertical depth of the partial encapsulation geomembrane, typically 5 to 10 feet according to Steinberg (1998).

- Method 1 (Picornell-Darder 1985):
    - Gather the information available for the climatic conditions and the soil properties for a specific site.
    - The design event is selected from a frequency analysis of the drought intensities that occur with a stochastic series of consecutive one-year meteorological events.
    - Either the deformation of the soil mass as a consequence of the moisture content changes is determined using a numerical simulation of the flow of water in or out of the region, or the water flux through the soil matrix is neglected and the deformation of the soil mass is estimated from the suction profile which is associated with the minimum depth of water stored in the soil profile in the design year.
    - The depth of the vertical moisture barrier extends to the zone of seasonal moisture changes in the soil mass.
- Method 2 (Lord and Koerner 1986):

- Based on the diffusion coefficient of soil and the soil half-space model, the general results are obtained as a function of membrane diffusion coefficient for the change in water content of an encapsulated soil mass over a given time.
- The designer selects the criteria for the allowable moisture change within the encapsulated soil for a specified time.
- Then the thickness and the diffusion coefficient of the membrane are determined.

Step 2. For pavement design, by maintaining constant moisture over time, the resilient modulus can be assumed to remain constant over the life of the pavement.

When partial encapsulation is used, pavement designers are cautioned regarding the use of the Enhanced Integrated Climatic Model (EICM) in the M-E design to predict the change in moisture over time.

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9.4 Geosynthetics – Overview of Construction Specifications and Quality Assurance

9.4.1 Specification Development

Method approach or performance approach specifications may be used for partial encapsulation projects. Method approach specifications state a specific installation pattern, procedure, and equipment. Method approach specifications should only be used if the specifying agency is confident in their understanding of partial encapsulation and its implementation methods. For this reason, performance approach specifications are typically

used. Performance approach specifications grant the contractor flexibility when selecting implementation methods which satisfy specified performance criteria and allocate most of the risk to the contractor.

For either specification, the contractor has to submit the details of the geomembrane, such as manufacturer, product name, composition, strength properties. The owner has to check the material and work done by the contractor as mentioned in the specification. Guidelines for geomembrane specifications are provided by FHWA (2008) with helpful commentary which allows the specification to be modified to address a project's particular needs during the preparation of guide specification. The specification should be complete such that the contractor can bid on the work without needing additional information. The specification should not require overly elaborate or expensive construction methods. The specification should contain all the detailed requirements necessary for the quality assurance, as appropriate to the technology and specification type. The specification should require the contractor to submit a detailed quality assurance plan for the installation of the liner. The specification should also contain information, such as minimum contractor qualifications, preconstruction meeting, etc. The geomembrane installer should be certified by the International Association of Geosynthetic Installers.

9.4.2 Summary of Quality Assurance

Quality control during the installation of the geomembrane is crucial to the performance of the product. Quality assurance methods for partial encapsulation projects include determination of the moisture variation of the encapsulated soil layer at different time periods from the construction phase to the end of the design life are necessary to evaluate the effectiveness of the partial encapsulation technique. During construction, the optimum moisture content and maximum dry density of the soil should be determined by the ASTM or AASHTO standards. These parameters can be monitored by sand cone test and/or nuclear gauge measurement. Either field CBR tests or Light Falling Weight Deflectometer (LFWD) could also be performed at the mid-depth of the fill and the surface to establish design parameters. Pavement performance should be monitored after construction including surface elevation changes that may be related to moisture content changes.

9.4.3 Summary of Instrumentation Monitoring and Construction Control

Instrumentation for monitoring moisture variation of the encapsulated soil layer includes techniques such as GPR and Time Domain Reflectometer (TDR) (ASTM D6565). Measurement of future shrinkage or swell could be made on the roadway surface using level surveys or the Benkelman beam test. Strain gages could also be attached to the liner to

indicate movement due to changes in moisture. FWD tests could also be performed on a periodic basis to monitor pavement performance and changes in subgrade support conditions.

9.5 Geosynthetic Cost Data

9.5.1 Cost Components

The primary contract pay items over and above normal subgrade construction and units of measurement used for encapsulation are the geomembrane measured by the square yard in-place, the excavation and replacement costs including flowable fill, if required, as measured in cubic yards. The unit price for installed geomembranes based on 2009 and 2010 bid prices was on the order of \$2.50 to \$10/yd^2, with bitumen coated geotextiles generally in the lower half of that range. Geomembrane specifications (weight, strength, etc.) have a large influence on material costs. Specialized applications such as bridge approaches and isolated embankments have higher unit costs than mass applications such as encapsulating a complete section of the roadway subgrade.

The equipment used to construct partial encapsulation is common on highway construction projects; therefore, additional mobilization costs are negligible. Production rates for the installation of membranes for partial encapsulation are controlled by related construction activities (e.g., placement of base course(s) and paving). Equipment and labor resources are easily adjusted to match the production rate of controlling activities with little effect on total cost. Additional information is available at *GeoTechTools*.

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Chapter 10

REINFORCED SOIL STRUCTURES

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1.0 DESCRIPTION AND HISTORY

1.1 Introduction

Several soil reinforcement technologies are addressed within this chapter. The following soil reinforcement technologies are presented in this chapter:

- Reinforced embankments over soft soils
- Reinforced soil retaining walls
- Reinforced soil slopes
- Soil nail walls

These are the principal reinforced soil technologies used in highway works. Related technologies are listed and described at the end of this chapter.

1.2 Description, Historical Overview, Focus, and Scope

Inclusions have been used since prehistoric times to improve soil. The use of straw to improve the quality of adobe bricks dates back to earliest human history. Many primitive people used sticks and branches to reinforce mud dwellings. Some other early examples of man-made soil reinforcement include dikes of earth and tree branches, which have been used in China for at least 1,000 years. During the 17th and 18th centuries, French settlers along the Bay of Fundy in Canada used sticks to reinforce mud dikes. Other examples include wooden pegs used for erosion and landslide control in England. Soil reinforcing can also be achieved by using plant roots.

Modern day applications use steel or geosynthetic reinforcements. Steel reinforcements are generally galvanized and are in strip, bar mat, welded wire mesh (WWM), or woven wire mesh form. Polymeric reinforcements are in geotextile, geogrid, geocomposite or geostrip form. Generally, these geosynthetics are polyester (PET), high density polyethylene (HDPE), or polypropylene (PP) based. PET fiber based geogrids and geostrips are coated with an acrylic, polyvinyl chloride (PVC), or polyethylene (PE) to hold the fibers in the grid or strip form and to provide protection during the installation process.

The scope of coverage within this chapter is presentation of abridged guidance and discussions on construction and materials, design, specifications, quality assurance, and costs. Overview discussion and guidance is provided for these technologies because detailed guidance and discussions are provided in other FHWA Geotechnical Engineering Circulars (GECs)/Reference Manuals. See Section 1.4 for key references, where detailed guidance is provided.

1.2.1 Reinforced Embankments

Embankments over soft soils (EOSS) have several potential failure modes. Rotational slope stability failure through the embankment and foundation can occur. Soil reinforcement placed at the embankment to foundation soil interface can be used to increase the resistance and stabilize the structure. Embankments constructed over soft foundation soils also have a tendency to spread laterally because of horizontal earth pressures acting within the embankment. These earth pressures cause horizontal shear stresses at the base of the embankment that must be resisted by the foundation soil. If the foundation soil does not have adequate shear resistance, failure can result. Properly designed horizontal layers of high-strength geotextile or geogrid reinforcements can also be used to increase lateral sliding stability and prevent such failures. Fundamentally, the reinforced soil at the base of the embankment is equivalent to a flexible mat foundation, preventing failure through the embankment and spreading the load over full width of the embankment.

Reinforcement is one of several technologies that may be used to design and construct embankments over soft soils. These other foundation treatment alternatives for the stabilization of embankments on soft or problem foundation soils should be carefully considered during the preliminary design phase. See other chapters of this reference manual for detailed information on other applicable technologies. See *GeoTechTools* for guidance on technology selection for this application.

The engineering of reinforced embankments over soft soils is very well established. Detailed FHWA guidance is provided in FHWA (2008a). Comprehensive training on the design and construction of reinforced embankments is provided in FHWA National Highway Institute (NHI) #132013A and #132013D courses.

1.2.2 Reinforced Soil Walls

The modern methods of soil reinforcement for retaining wall construction were pioneered by the French architect and engineer Henri Vidal in the early 1960s. His research led to the invention and development of Reinforced Earth®, a system in which steel strip reinforcements are placed horizontally in the soil in multiple, equally spaced layers and attached to metallic and later precast concrete facing panels. The first wall to use this technology in the United States was built in 1972 on California State Highway 39, northeast of Los Angeles. It is estimated that more than 9,000,000 square feet of MSE retaining walls with precast facing are constructed on average every year in the United States (GEC 11). The tallest wall constructed in the United States is 150 feet high. Since the introduction of Reinforced Earth®, a number of other proprietary and nonproprietary systems have been developed and used throughout the United States.

The use of geotextiles in MSE walls started after the beneficial effect of reinforcement with geotextiles was noticed in highway embankments constructed over weak soils. The first geotextile-reinforced wall was constructed in France in 1971 (Puig et al. 1977), and the first structure of this type in the United States was constructed in 1974 (USDA 1977). Geogrids for soil reinforcement were developed around 1980. The first use of geogrid in earth reinforcement was in 1981. Extensive use of geogrid products in the United States started in about 1983, and they now comprise a large portion of the market. Since the early 1980s, the use of geosynthetics in reinforced soil wall structures has increased significantly. It is estimated that more than 3,000,000 square feet of modular block wall (MBW) faced, geosynthetic reinforced MSE walls are constructed yearly in the United States when considering all types of transportation related applications (GEC 11).

The engineering of reinforced soil or mechanically stabilized earth (MSE) walls is very well established. Detailed FHWA guidance is provided in GEC 11 and in AASHTO (2014). Comprehensive training on the design and construction of MSE walls is provided in FHWA NHI #132042, #132043 and #132080 courses.

1.2.3 Reinforced Soil Slopes

Reinforced soil slopes as with MSE walls, consist of multiply spaced, horizontal reinforcement layers placed between compacted soil lifts, but in a constructed earth slope. The reinforcements are extended to the face of the slope and attached to a facing system, often vegetated, to prevent erosion of the face. The first reported use of reinforced steepened slopes (RSS) is believed to be the west embankment for the Great Wall of China. The tallest successfully designed, constructed and in-service RSS structure in the United States to date is 141 feet high. The use of RSS structures has expanded dramatically over the last few decades. As of 2009, more than about 150 RSS projects were being constructed yearly in connection with transportation related projects in the United States and the number is continuing to grow.

Geosynthetic and steel WWM soil reinforcements are used with RSS structures. The engineering of reinforced soil slopes is very well established. Detailed FHWA guidance is provided in GEC 11. Comprehensive training on the design and construction of reinforced soil slopes on firm foundations is provided in FHWA NHI #132042 and #132043 courses.

1.2.4 Soil Nail Walls

One of the first applications of this technology in the United States was in 1976 when soil nails were used to provide support to a 45-foot deep excavation for expansion of the Good Samaritan Hospital in Portland, Oregon. It was estimated that this system was completed in nearly half the time and at about 85 percent of the cost of conventional excavation-support

systems (FHWA 1998). In 1984 FHWA funded a demonstration project for the installation of a prototype, 40-foot high soil nail wall near Cumberland Gap, Kentucky (Nicholson 1986).

Since its introduction in the United States, the use of soil nail walls has increased greatly for roadway projects. This increase can be attributed to the technical feasibility and cost-competitiveness of soil nailing. For certain subsurface and project conditions, soil nailing is more advantageous than other top-down, earth retaining systems because the construction equipment is smaller, and it provides greater structural redundancy. Easements tend to be smaller for soil nail projects because soil nails are shorter than ground anchors, for example, given the same wall height. Additionally, as the use of soil nailing has grown, the number of qualified, soil-nail specialty contractors has increased. (GEC 7)

The engineering of soil nail walls is very well established. Detailed FHWA guidance is provided in GEC 7. Comprehensive training on the design and construction of soil nail walls is provided in FHWA NHI #132085 course.

1.2.5 Related Technologies

There are several other technologies that are related to or are an extension of the four primary reinforcement applications of embankments over soft soils, MSE walls, reinforced soil slopes, and soil nail walls. These technologies are listed and described in Section 6.0.

1.3 Glossary

A variety of terms are used with reinforced soil technologies. For clarity, they are defined within this manual as:

Inclusion – a generic term that encompasses all man-made elements incorporated in the soil to improve its behavior. Examples of inclusions are steel strips, geotextile sheets, steel or polymeric grids, steel nails, and steel tendons between anchorage elements.

Reinforcement – term used only for those *inclusions* where soil-inclusion stress transfer occurs continuously along the inclusion, i.e. soil reinforcement.

Reinforced Soil – term used when multiple layers of inclusions act as reinforcement in soils.

Reinforced Fill – the zone of where the reinforcements are placed in a soil fill material.

Retained Backfill – the fill or in situ material located behind the reinforced soil zone.

Mechanically Stabilized Earth Walls (MSE wall or MSEW) – a form of reinforced soil that incorporate planar reinforcing elements in fill constructed earth structures with face inclinations of near vertical to slight batter (20º or less).

Reinforced Soil Slopes (RSS) – a form of reinforced soil that incorporates planar reinforcing elements in earth-sloped structures with face inclinations of less than 70 degrees.

Geosynthetics – a generic term that encompasses flexible polymeric materials used in geotechnical engineering such as geotextiles, geogrids, and geostraps.

Facing – the component of the reinforced soil system used to prevent the soil from raveling out between the rows of reinforcement. Common facings include precast concrete panels, dry cast modular blocks, gabions, welded wire mesh, shotcrete, timber lagging, polymeric cellular confinement systems, and wrapped sheets of geosynthetics. The facing also plays a minor structural role in the stability of the structure. For RSS structures it usually consists of welded wire mesh, geosynthetic wrap-around, and/or some type of erosion control material.

GRS – is a term for geosynthetic reinforced soil (GRS) that has closely spaced soil reinforcement layers (in vertical direction) in fill constructed earth structures. FHWA has developed an approach (FHWA 2011a) for design of geosynthetic reinforced soil walls based upon composite action, in lieu of designing reinforcements as inclusions.

Soil Nails – passive reinforcing elements that are drilled and grouted sub-horizontally in the ground to support excavations in soil, or in soft and weather rock that: contribute to the stability of earth-resisting systems mainly through tension as a result of the deformation of the reinforced soil or weathered rock mass; transfer tensile loads to the surrounding ground through shear stresses along the grout-ground interface; and interact structurally with the facing (after GEC 7).

A variety of acronyms are used with reinforced soil technologies. For clarity, they are defined as follows throughout this manual:

- **EOSS** embankment over soft soils
- **HDPE** high density polyethylene
- **MSE** mechanically stabilized earth
- **MSEW** mechanically stabilized earth wall
- **PE** polyethylene
- **PET** polyester
- **PP** polypropylene
- **PVC** polyvinyl chloride

- **RSS** reinforced soil slope

1.4 Primary References

- AASHTO. (2014). *LRFD Bridge Design Specifications*. 7th Edition, with 2015 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO. (2010). *LRFD Bridge Construction Specifications*, 3rd Edition, with 2011, 2012, 2013, 2014, and 2015 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, D.C.
- FHWA. (2008). *Geosynthetic Design and Construction Guidelines*. Authors: Holtz, R.D., Christopher, B.R., and Berg, R.R., Federal Highway Administration, U.S. DOT, Washington, D.C., FHWA-HI-07-092, 460p.
- GEC 7. (2015). *Soil Nail Walls Reference Manual*. Authors: Lazarte, C.A., Robinson, H., Gómez, J.E., Baxter, A., Cadden, A., and Berg, R.R. FHWA NHI-14-007, Federal Highway Administration, U.S. DOT, Washington, D.C., 425p.
- GEC 11. (2009). *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*. Authors: Berg, R.R., Christopher, B.R., and Samtani, N.C., FHWA NHI-10-024 Vol I and NHI-10-025 Vol II, Federal Highway Administration, U.S. DOT, Washington, D.C., 306p. (Vol I) and 378p. (Vol II).

2.0 REINFORCED EMBANKMENTS

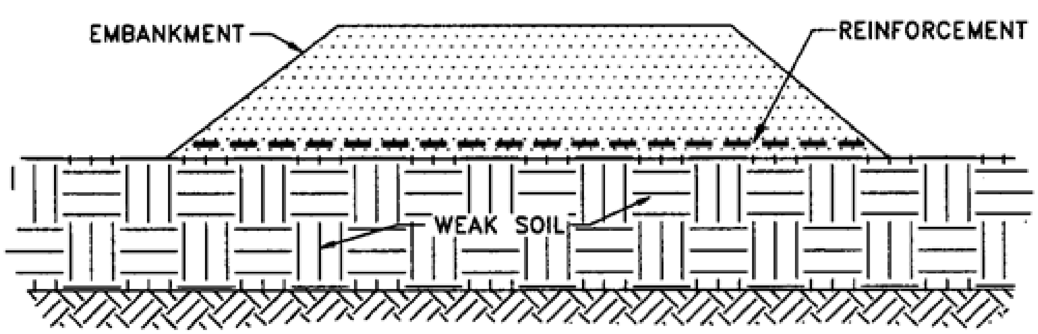
2.1 Feasibility Considerations

Reinforced EOSS structures are cost-effective alternatives for many applications where the foundation soils are too weak to support the weight and geometry of the planned embankment. One, or more, layers of high strength geotextile or geogrid are generally used at the embankment to foundation interface to reinforce the structure.

Reinforced EOSS offer significant technical and cost advantages over the conventional excavate and replace soft soils technique. However, while a reinforced EOSS may reduce the displacement of foundation soil, it will not decrease the amount of total anticipated embankment settlement or time to achieve primary consolidation. Other techniques or combination of techniques may provide the best option for a particular project, especially where the rate and/or magnitude of settlement is of concern. For example, reinforced embankments have been used in combination with the prefabricated vertical drains (see Chapter 2) technology increase rate of settlement and with lightweight fills (see Chapter 3) to decrease total settlement.

2.2 Applications

The common application an embankment, dike, or levee constructed over very soft, saturated silt, clay, or peat layers is illustrated in Figure 10-1. In this situation, the reinforcement is usually placed with its strong direction perpendicular to the centerline of the embankment, and plane strain conditions are assumed to prevail. Generally, the reinforcement provides stability of the embankment until the underlying soft soils consolidate, gain strength, and are capable of supporting the embankment. Additional reinforcement with its strong direction oriented parallel to the centerline may also be required at the ends of the embankment (e.g., at a bridge abutment).



After Bonaparte and Christopher 1987

Figure 10-1. Common reinforced embankment application.

In addition to the reinforcement function, geotextiles may also be used as separators for displacement-type embankment construction (Holtz, 1989) and as stabilization layers to support embankment construction equipment or ground modification equipment (see Chapter 9). In this application, the geotextile does not provide significant reinforcement but acts primarily as a separator to maintain the integrity of the embankment as it displaces the subgrade soils. In this case, geotextile design is based upon constructability, survivability, and filtration requirements (i.e., to promote foundation drainage) and a high elongation material may be selected.

2.2.1 Advantages and Potential Disadvantages

2.1.2.1 Advantages

Geosynthetic reinforced EOSS have many advantages compared with conventional excavate and replace technique and when compared to other alternative technologies. Reinforced embankments provide a:

- Increase in the design global factor of safety, and height of the embankment
- Reduction or elimination of stabilizing side berms, thus reducing fill requirements
- Reduction in displacement of foundation soil during construction, potentially reducing fill requirements
- Reduction in differential settlement
- Most general contractors can construct GREs and specialty contractors are not required

2.1.2.2 Potential Disadvantages

The following are potential disadvantages that may be associated with a reinforced embankment project:

- The magnitude of total settlement will likely not be reduced
- The time to achieve primary consolidation is not reduced
- Detailed field observations are generally required during construction to monitor pore pressures, total settlement, and rate of settlement

2.2.2 Feasibility Evaluations

Many options should be considered when encountering problem foundations for a proposed highway embankment including an initial evaluation of three basic choices of: (i) a not build or realignment alternative, (ii) place the roadway on a structure, and (iii) use some type of

foundation soil modification. Problem foundation soils can be improved by (Broms 1979): (i) reducing the load, (ii) replacing the poor soils with better materials, (iii) increasing the shear strength and reducing the compressibility of the problem soils, (iv) transferring the loads to more competent, and (iv) reinforcing the embankment and/or its foundation. (Holtz 1989)

Holtz (1989) discusses the above treatment alternatives and provides guidance about when embankment reinforcement is feasible. Other ground modification options and their feasibility, advantages, potential disadvantages, and limitations are presented in other chapters of this reference manual. The following discussions assume that alternative foundation treatments have been considered and that a reinforced embankment has been selected.

2.1.3.1 Geotechnical

A reinforced embankment can be constructed over any depth of soft soils, unlike other options that have depth limits. If the depth of soft soils is small relative to the width of the embankment, an additional stability mode of lateral squeeze failure should be examined.

The development and implementation of a comprehensive subsurface investigation program is a key element for ensuring successful project design and construction. High quality, undisturbed soil samples are required for determination of engineering properties. Settlement analyses must assess primary, secondary, total and differential settlements, both transversely and longitudinally along across the embankment.

2.1.3.2 Environmental Considerations

Embankments on soft soils are often constructed in wetlands. The selection of an option for embankment construction, and the design of a reinforced soil embankment option, may be impacted by the allowable width of wetland disturbance.

Another consideration in the selection of embankment construction option is whether or not the foundation soils and/or groundwater are contaminated. Furthermore, if the soils are contaminated, can they remain in-place, undisturbed or if removed, do they needed to be treated? A reinforced embankment leaves the underlying foundation soils in-place, and caps the foundation soils with the embankment fill.

2.1.3.3 Site Conditions

The existing vegetation, groundwater level, and strength of the underlying foundation soils will dictate construction methods for the initial lifts of the embankment. See FHWA (2008a)

for a discussion of site clearing, geosynthetic reinforcement placement, and embankment fill placement and compaction techniques for reinforced soil embankments.

2.2.3 Limitations

The total settlement magnitude will not be significantly reduced from the unreinforced embankment option. Generally a field instrumentation and monitoring program is required to measure pore pressures in the foundation soils, maintain adequate safety factors, and oversee the rate of embankment fill placement. When used alone, this technology is not appropriate for projects that cannot accommodate the time necessary for consolidation of underlying foundation soils.

2.2.4 Alternative Solutions

Reinforcement is one of many technologies that can be used to design and construct EOSS. If the soft soils only extend to a relatively shallow depth, excavation and replacement is a typical and cost-effective alternative. Other ground modification alternatives are:

- prefabricated vertical drains and fill preloading (see Chapter 2)
- deep dynamic compaction (see Chapter 4)
- vibro-compaction (see Chapter 4)
- lightweight fills (see Chapter 3)
- column supported embankments, with or without a load transfer platform (see Chapter 6)

Applicability of reinforcement and of these alternatives is dependent upon the type, depth and strength of the foundation soil(s). Additionally, project constraints and risks should be factored into selection of a technology for a particular project.

2.3 Construction and Materials

General construction requirements and guidance for geosynthetic reinforced embankments are presented in FHWA (2008a). The general sequence of reinforced embankment construction is presented below. Primary materials used in construction are the geosynthetic soil reinforcement and the embankment fill soils.

2.3.1 Construction

The construction procedures for reinforced embankments on soft foundations are extremely important. Improper fill placement procedures can lead to geosynthetic damage, nonuniform

settlements, and even embankment failure. With the use of low ground pressure equipment, a properly selected geosynthetic, and proper procedures for placement of the fill, these problems can essentially be eliminated. Primary construction steps are listed below. The essential construction details for each step are addressed in FHWA (2008a).

1. Prepare subgrade, leaving tree stumps, grass, and root mass in-place to provide construction equipment support.
2. Geosynthetic placement procedures, including orientation, roll width, roll lengths, sewing requirements, manual tension requirements, and inspection and repair techniques.
3. Fill placement, spreading, and compaction procedures including fill placement sequence, initial lift height and compaction requirements, and subsequent lift heights and compaction requirements. Fill placement procedures vary between construction on extremely soft foundations from those used on moderate ground conditions.
4. Construction monitoring with piezometers, settlement plates, and fill thickness measurements.

2.3.2 Geosynthetic Reinforcement

The geosynthetic strength requirements, including seam strength, are quantified in the analysis and design process. In addition to tensile and frictional properties, geosynthetic specification must consider drainage requirements, construction conditions, and environmental factors. Geosynthetic properties required for reinforcement applications are given in Table 10-1. See FHWA (2008a) for discussions on the geosynthetic reinforcement property requirements, and for guidance on construction survivability requirements and project conditions.

Table 10-1. Geosynthetic Properties Required for Reinforced Embankments

Criteria	Parameter	Property
Design Requirements: Mechanical	Tensile strength	Wide width strength
Design Requirements: Mechanical	Tensile modulus	Wide width strength
Design Requirements: Mechanical	Seam strength	Wide width strength
Design Requirements: Mechanical	Tension creep	Tension creep
Design Requirements: Mechanical	Soil-geosynthetic friction	Soil-geosynthetic friction angle
Design Requirements: Hydraulic	Piping resistance	Apparent opening size
Design Requirements: Hydraulic	Permeability	Permeability
Constructability Requirements	Tensile strength	Grab strength
Constructability Requirements	Puncture resistance	Puncture resistance
Constructability Requirements	Tear resistance	Trapezoidal tear
Longevity	UV stability (if exposed)	UV resistance
Longevity	Soil compatibility (where required)	Chemical; Biological

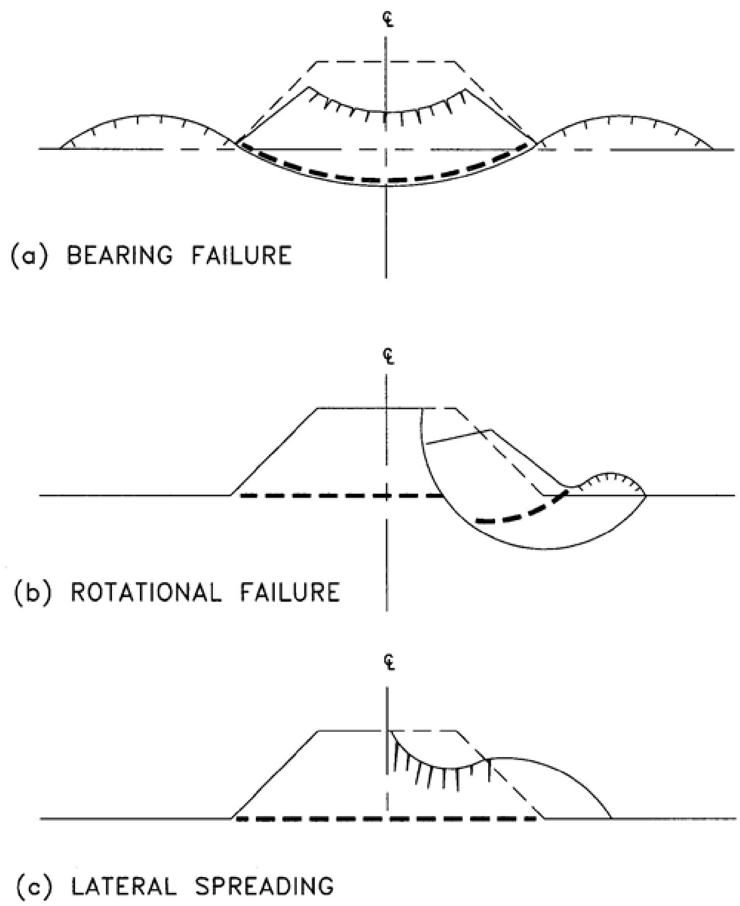
2.3.3 Embankment Fill

The first lift of fill material just above the geosynthetic reinforcement should be a free-draining granular material. This material serves as a drainage layer for excess pore water dissipating from the underlying soils requirement, and provides the good frictional interaction between the geosynthetic and embankment fill. Other lower permeability, (preferably granular) fill materials may be used above this layer as long as the strain compatibility of the geosynthetic is evaluated with respect to the fill material. If lower

permeability fill is used, horizontal prefabricated geocomposite drains (e.g., strip drains) placed directly on the foundation soil should also be considered to provide drainage and facilitate consolidation.

2.4 Design Overview

As with ordinary embankments on soft soils, the basic design approach for reinforced embankments is to design against failure. Potential failure modes of a reinforced embankment are illustrated in Figure 10-2. The three possible modes of failure indicate the types of stability analyses that are required. Additionally, settlement of the embankment and potential creep of the reinforcement must be considered, although creep is only a factor if the creep rate in the reinforcement is faster than the rate of strength gain occurring in the underlying foundation soils due to consolidation. Because the most critical condition for embankment stability is at the end of construction, the reinforcement only has to function until the foundation soils gain sufficient strength to support the embankment. (FHWA 2008a)



FHWA 2008a, after Haliburton et al. 1978

Figure 10-2. Reinforced embankment failure modes.

2.4.1 Design Considerations

The calculations required for stability and settlement utilize conventional geotechnical design procedures modified only for the presence of the reinforcement. The stability of an embankment over soft soil is usually determined by the *total stress* method of analysis, which is conservative since the analysis generally assumes that no strength gain occurs in the compressible soil. The stability analyses presented in FHWA (2008a) use the *total stress* approach, because it is simple and appropriate for reinforcement design (Holtz 1989). The *total stress* design steps and methodology are summarized in the following section.

It is always possible to calculate stability in terms of the effective stresses using the *effective stress* shear strength parameters. However, this calculation requires an accurate estimate of the field pore pressures to be made during the project design phase. Additionally, high-quality, undisturbed samples of the foundation soils must be obtained and K_o consolidated-undrained triaxial tests conducted in order to obtain the required design soil parameters. Because the prediction of in situ pore pressures in advance of construction is not easy, it is essential that field pore pressure measurements using high quality piezometers be made during construction to control the rate of embankment filling. Note that by taking into account the strength gain that occurs with controlled rate (e.g. staged) embankment construction, lower strength and therefore lower cost reinforcement can be utilized. However; the time required for construction may be significantly increased and the costs of the site investigation, laboratory testing, design analyses, field instrumentation, and inspection are also greater. (FHWA 2008a)

2.4.2 Design Steps

A step-by-step procedure for design of reinforced embankments based upon total stress analyses is listed in Table 10-2. Substeps and detailed commentary on each step can be found in FHWA (2008a).

Table 10-2. Reinforced Embankment Design Steps

Step	Description
Step 1.	Define embankment dimensions and loading conditions.
Step 2.	Establish the soil profile and determine the engineering properties of the foundation soil.
Step 3.	Obtain engineering properties of embankment fill materials.
Step 4.	Establish minimum appropriate factors of safety and operational settlement criteria for the embankment.
Step 5.	Check bearing capacity.
Step 6.	Check rotational shear stability.
Step 7.	Check lateral spreading (sliding) stability.
Step 8.	Establish tolerable geosynthetic deformation requirements and calculate the required reinforcement modulus, J, based on wide width (ASTM D4595) tensile testing.
Step 9.	Establish geosynthetic tensile strength requirements in the embankment's longitudinal direction (i.e., direction of the embankment alignment).
Step 10.	Establish geosynthetic modulus, seam strength, soil-geosynthetic friction, and survivability and construction properties.
Step 11.	Estimate magnitude and rate of embankment settlement.
Step 12.	Establish construction sequence and procedures.
Step 13.	Establish construction observation and monitoring requirements.
Step 14.	Hold preconstruction meetings.
Step 15.	Observe construction and build with confidence.

2.4.3 Primary Design References

The primary reference for total stress design of reinforced embankments is:

- FHWA. (2008a). *Geosynthetic Design and Construction Guidelines*. Authors: Holtz, R.D., Christopher, B.R., and Berg, R.R., Federal Highway Administration, U.S. DOT, Washington, D.C., FHWA-HI-07-092, 460p.

Additional references include FHWA (1985); Bonaparte and Christopher (1987); and the Canadian Foundation Engineering Manual (2006). See Li and Rowe (2001) for an analysis method where prefabricated vertical drains (see Chapter 2) are combined with geosynthetic reinforcement for embankment construction. For reinforced embankments over fibrous peats, see Rowe and Soderman (1985) for peat underlain by a firm base and Rowe and Soderman (1986) for peat underlain by soft cohesive layer.

2.5 Overview of Construction Specifications and Quality Assurance

2.5.1 Specification Development

Because the reinforcement requirements for soft-ground embankment construction will be project and site specific, standard specifications, which include suggested geosynthetic properties, are not appropriate, and special provisions or a separate project specification must be used. An example reinforced embankment specification that includes most of the items which should be considered in a project is presented in FHWA (2008a). This specification is from the Washington Department of Transportation and is a method type specification.

2.5.2 Summary of Quality Assurance

The construction procedures used by a contractor are crucial to the success of a reinforced embankment on a very soft foundation project. Therefore, competent and professional construction inspection is absolutely essential. Field personnel must be properly trained to observe every phase of the construction and to ensure that (1) the specified reinforcement material is delivered to the project, (2) the geosynthetic is not damaged during construction, and (3) the specified sequence of construction operations are explicitly followed.

The components of quality assurance programs for geosynthetic reinforced embankment projects are listed in Table 10-3 (*GeoTechTools*). The entries in the table are a list of typical items, not a list of all methods that could be used for quality assurance.

Table 10-3. Typical Components of Reinforced Embankment Quality Assurance

Topics	Results
Existing material related procedures	• Embankment soil shear strength • Soil reinforcement tensile seam strength
Existing process control procedures	• Monitor foundation pore water pressure • Monitor settlement and fill thickness • Monitor soil reinforcement placement geometries
Existing material related measurement values	• Soil shear strength testing • Soil reinforcement coupon testing • Soil reinforcement seam coupon testing • Soil reinforcement manufacturer's certification
Existing process control measurement values	• Monitor foundation pore water pressure • Monitor thickness of soil lifts • Monitor settlement and fill thickness • Monitor soil reinforcement placement geometries • Monitor soil reinforcement quantities
Material performance criteria	• Monitor soil reinforcement strain • Foundation soil shear strength gain over time
System behavior performance criteria	• Embankment settlement • Foundation pore water pressure • Lateral deformation response of embankment and foundation soil
Emerging process control procedures and measurement values	• Intelligent geosynthetic • Automated instrumentation

2.5.3 Summary of Instrumentation Monitoring and Construction Control

As noted in Table 10-2, one step in the design process of reinforced embankments is construction observation. This observation includes the construction quality assurance program, instrumentation installation, and monitoring of instrumentation. As a minimum, instrumentation should include the installation of piezometers, settlement points, and surface survey monuments. Inclinometers also may be installed to observe lateral movement with depth. The purpose of the instrumentation in soft ground reinforcement projects is not for research but to verify design assumptions and to control and, usually, expedite the

construction schedule (e.g., through a more accurate determination of the time required for settlement).

The first step in planning a monitoring program is to define the purpose of the measurements. Every instrument on a project should be selected and placed to assist in answering a specific question. *If there is no question, there should be no instrumentation*. Both the questions that need to be answered and the clear purpose of the instrumentation in answering those questions should be established. The most significant parameters of interest should be selected, with care taken to identify secondary parameters that should be measured if they may influence primary parameters.

2.6 Cost Data

The cost analysis for a geosynthetic reinforced embankment (FHWA, 2008a) includes:

1. Geosynthetic cost: including purchase price, factory prefabrication, and shipping.
2. Site preparation: including clearing and grubbing, and working table preparation.
3. Geosynthetic placement: related to field workability,
    a. with no working table, or
    b. with a working table.
4. Fill material: including purchasing, hauling, dumping, compaction, allowance for additional fill due to embankment subsidence. Use free-draining granular fill for the lifts adjacent to geosynthetic to provide good reinforcement interaction and drainage.

2.6.1 Cost Components

Typical contract pay items and units of measurement used for geosynthetic reinforced embankments include:

- Geosynthetic (fabric or grid) measured by the square yard in-place.
- Granular material measured by the ton.
- Embankment material measured by the cubic yard.

Geosynthetic reinforced embankments are constructed with operations that are common to highway construction projects, except with low ground pressure equipment (e.g., wide track dozers), which is not always common. Generally, mobilization costs are negligible.

3.0 REINFORCED SOIL WALLS

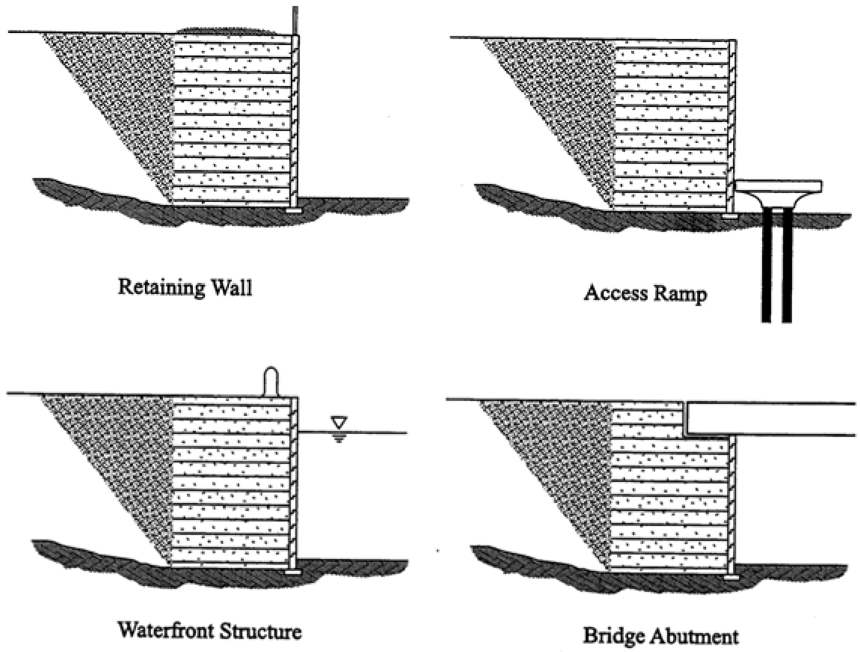
3.1 Feasibility Considerations

MSE wall (MSEW) structures are cost-effective alternatives for most applications where reinforced concrete or gravity type walls have traditionally been used to retain soil.

MSE walls also offer significant technical and cost advantages over conventional reinforced concrete retaining structures, especially at sites with poor foundation conditions. In such cases, the elimination of costs for foundation improvements such as piles and pile caps, that may be required for support of conventional structures, have resulted in cost savings of greater than 50 percent on completed projects.

3.1.1 Applications

Representative uses of MSE walls for various applications are shown in Figure 10-3. These include bridge abutments and wing walls, as well as areas where the right-of-way is restricted, such that an embankment or excavation with stable side slopes cannot be constructed. They are particularly suited to economical construction in steep-sided terrain, in ground subject to slope instability, or in areas where foundation soils are poor. A typical application of an elevated interstate in an urban area is shown in Figure 10-4. The tallest MSE wall in the United States to date has an exposed height of 138 feet at its tallest section and was constructed for the Third Runway project at SeaTac Airport, and is shown in Figure 10-5.



GEC 11

Figure 10-3. Representative MSE wall application: retaining wall (top left), access ramp (top right), waterfront structure (bottom left) and bridge abutment (bottom right).

GEC 11

Figure 10-4. MSE wall construction on MnDOT Crosstown Project.

GEC 11

Figure 10-5. SeaTac Airport runway extension MSE wall.

Temporary MSE wall structures have been especially cost-effective for temporary detours necessary for highway reconstruction projects. Temporary MSE walls are used to support temporary roadway embankments and temporary bridge abutments, as illustrated in Figure 10-6. MSE walls are also used as temporary support of permanent roadway embankments for phased construction, an example is shown in Figure 10-7.

GEC 11

Figure 10-6. MSE walls to support temporary bridge abutment and roadway embankment.

GEC 11

Figure 10-7. MSE wall used to temporarily support a permanent roadway embankment for phased construction.

3.1.2 Advantages and Potential Disadvantages

3.1.2.1 Advantages

MSE walls have many advantages compared with conventional reinforced concrete and concrete gravity retaining walls. MSE walls:

- Are significantly faster to construct due to simple, rapid construction procedures, do not require as large of construction equipment, and no waiting time required for concrete curing.
- Do not require special skills for construction.
- Require less site preparation than other alternatives.
- Need less space in front of the structure for construction operations.
- Reduce right-of-way acquisition.
- Do not need rigid, unyielding foundation support because MSE structures are tolerant to deformations.
- Are cost effective.
- Are technically feasible to heights in excess of 100 feet.

Pre-manufactured materials, rapid construction, and competition among different proprietary systems have resulted in a cost reduction relative to traditional types of retaining walls. MSE walls are likely to be more economical than other wall systems for walls higher than about 10 feet or where special foundations would be required for a conventional wall.

One of the greatest advantages of MSE walls is their flexibility and capability to tolerate deformations due to poor subsoil foundation conditions. Also, based on observations in seismically active zones, these structures have demonstrated a higher resistance to seismic loading than rigid concrete wall structures.

3.1.2.2 Potential Disadvantages

The following general potential disadvantages associated with MSE wall structures:

- Require a relatively large space (e.g., excavation if in a cut) behind the wall face to install required reinforcement.
- MSE walls require the use of select granular fill.
- The design of soil-reinforced systems often requires a shared design responsibility between material suppliers and owners.
- No dig zones within the reinforced section (e.g., for installation or repair of utilities)

3.1.3 Feasibility Evaluations

3.1.3.1 Geotechnical

The development and implementation of an adequate subsurface investigation program for the existing foundation conditions is a key element for ensuring successful project implementation. Causes for distress experienced in projects are often traced to inadequate subsurface exploration programs that did not disclose local or significant areas of soft soils, causing significant local differential settlement and distress to the wall facing. In a few documented extreme cases, such foundation weakness caused complete foundation failures leading to catastrophic collapses.

Determination of engineering properties for foundation soils should be focused on establishment of bearing resistance, global stability, settlement potential, and position of groundwater levels. The adequacy of the foundation to support the fill weight must be determined as a first-order feasibility evaluation. Where soft compressible soils are encountered, preliminary stability analyses must be made to determine if sufficient shear strength is available to support the weight of the reinforced wall fill. As a rough first approximation for MSE wall structures, the available shear strength must be equal to at least

2.0 to 2.5 times the weight of the fill structure. Where these conditions are not satisfied, ground improvement techniques must be considered to increase the bearing capacity at the foundation level.

Where marginal to adequate foundation strength is available, preliminary settlement analyses should be made to determine the potential for differential settlement, both longitudinally along a proposed structure as well as transverse to the face. This second-order feasibility evaluation is useful in determining the appropriate type of facing systems for MSE walls and in planning appropriate construction phasing to accommodate the settlement.

3.1.3.2 Environmental Considerations

The primary environmental condition affecting reinforcement type selection and potential performance of MSE structures is the aggressiveness of the in-situ ground regime that can cause deterioration to the reinforcement. Post construction soil environment changes must be considered where deicing salts or fertilizers are subsequently used.

For steel reinforcements, in situ regimes containing chloride and sulfate salts generally in excess of 200 ppm accelerate the corrosive process as do acidic regimes characterized by a pH of less than 5 (FHWA 2009). Alkaline regimes characterized by $pH > 10$ will cause accelerated loss of galvanization.

Certain in situ regimes have been identified as being potentially aggressive for geosynthetic reinforcements. Polyester (PET) degrades in highly alkaline or acidic regimes. Polyolefins appear to degrade only under certain highly acidic conditions where metals are present (e.g., mine waste). For additional specific discussions on the potential degradability of reinforcements, refer to Corrosion/Degradation reference manual, FHWA (2009).

The primary environmental condition affecting facing type selection and potential performance of MSE structures is the likelihood of deicing salt spray on the wall face. Dry-cast concrete MBW facing units are susceptible to freeze-thaw degradation with exposure to deicing salts and cold temperatures. This is a concern in northern tier states that use deicing salts. Some vendors have developed special mix designs, with additive(s), and manufacturing processes that result in units that are very durable and resistant to freeze-thaw degradation. See GEC 11 for detailed discussion on this issue and specification recommendations.

3.1.3.3 Site Conditions

MSE wall structures are particularly well suited where a "fill-type" wall must be constructed or where side-hill fills are indicated. Under these latter conditions, the volume of excavation may be small, and the general economy of this type of construction is not jeopardized.

Economic advantages diminish with large cut volumes to accommodate the reinforced soil structure, but in many instances remain viable.

A secondary issue may be site accessibility, which could dictate the nature and size of the facing for MSE wall construction. Sites with poor accessibility or remote locations may lend themselves to lightweight facings such as geotextile or geogrid wrapped facings and vegetative covers; metal skins; welded wire mesh, gabions, or MBW units which could be erected without heavy lifting equipment.

3.1.3.4 Aesthetics

Precast concrete facing panels may be cast with an unlimited variety of texture and color for an additional premium that seldom exceeds 15 percent of the facing cost, which on average would mean a 4 to 6 percent increase on total in-place cost. Modular block wall facings are often comparable in cost to precast concrete panels except on small projects (less than 4,000 square feet) where the small size introduces savings in erection equipment cost and the need to cast special, made-to-order concrete panels to fit what is often irregular geometry. MBW facings may be manufactured in color and with a wide variety of surface finishes. (GEC 11)

3.1.4 Limitations

The current AASHTO LRFD Bridge Design Specifications (2014) states that MSE walls should not be used under the following conditions:

- When utilities other than highway drainage are to be constructed within the reinforced zone unless access is provided to utilities without disrupting reinforcements and breakage or rupture of utility lines will not have a detrimental effect on the stability of the structure.
- Where floodplain erosion or scour may undermine the reinforced fill zone or facing, or any supporting footing.
- With reinforcements exposed to surface or ground water contaminated by acid mine drainage, other industrial pollutants, or other environmental conditions defined as aggressive in Article 7.3.6.3 of the AASHTO LRFD Bridge Construction Specifications (2010), unless environmental-specific, long-term corrosion, or degradation studies are conducted.

3.1.5 Alternative Solutions

Cantilever, gravity, semi gravity, or counterfort concrete walls or soil embankments are the usual alternatives to MSE walls and abutments. Embankments may be conventional soil slopes or reinforced soil slopes (see Section 4.0 Reinforced Soil Slopes).

In cut situations, in situ walls such as tieback anchored walls, soil nailed walls or nongravity cantilevered walls are often more economical, although where limited ROW is available, a combination of a temporary or permanent in situ wall at the back end of the reinforcement and a permanent MSE wall is often competitive. Use of a permanent in situ wall at the back of the MSE wall will allow for much shorter reinforcement in the MSE wall (see Section 6.2 and GEC11 for additional information and design guidance.)

For waterfront or marine wall applications, sheetpile walls with or without anchorages that can be constructed in the wet are often, if not always, both more economical and more practical to construct.

3.2 Design Overview

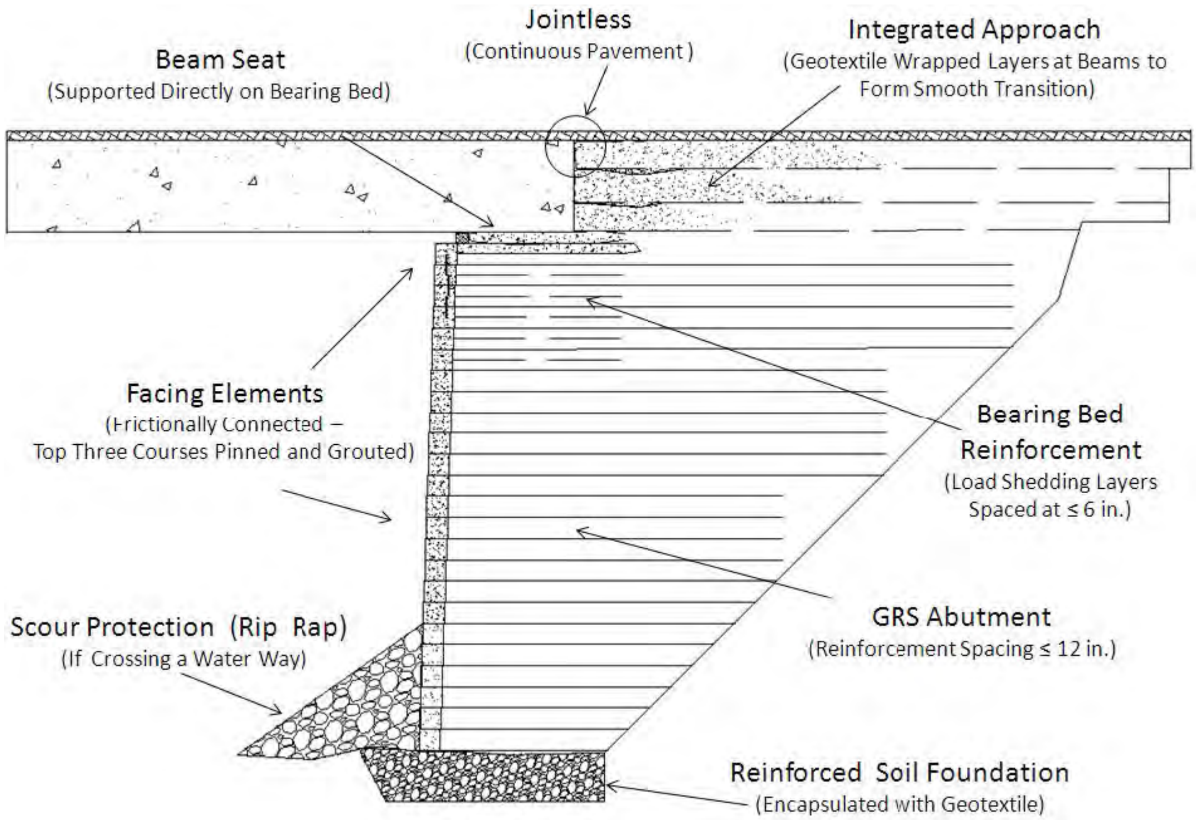
MSE walls are generally designed as a soil mass with discrete soil reinforcement modeled as tension inclusions. The principal methodology used for design is known as the *Simplified Method*. This method was first established as an allowable strength design (ASD) methodology and was updated to its current load and resistance factor design methodology, in the primary references listed below. The Simplified Method is widely used for transportation walls that are typically near vertical in batter, generally have uniform soil reinforcement lengths, and use a select granular reinforced soil fill. The internal stability of the reinforced mass is independent of the wall facing element with this design procedure. AASHTO (2014) and GEC11 (2009) do provide guidance for factoring deep facing elements into external stability analysis, significantly battered walls, and non-uniform reinforcement lengths into design.

The simplified method is applicable to steel strip, steel mat, geogrid, and geotextile soil reinforcements. As noted in AASHTO (2014), internal reinforcement loads for steel reinforced walls may be computed with the *Coherent Gravity Method* in lieu of the simplified method. Currently, neither AASHTO (2014) nor GEC 11 provide design guidance for the use of geosynthetic strap reinforcements.

Alternatively to designing the reinforcements as an inclusion, the reinforced soil and reinforcements may be designed as a composite mass. FHWA has developed an approach (FHWA 2011a) for designing closely spaced geosynthetic reinforced soil walls based upon

composite action. In recent years this method has been used for design of abutments supporting bridges, and has been integrated into design of single (generally) span bridges.

Note that the when closely spaced soil reinforcement system is combined with an open-graded aggregate reinforced fill and a reinforced soil foundation, and is used for bridge abutment support it is known as a geosynthetic reinforced soil – integrated bridge system (GRS-IBS). The GRS-IBS, illustrated in Figure 10-8, is commonly used in abutment support applications for single span bridges over creeks and streams.



FHWA 2011a

Figure 10-8. Typical cross section of a geosynthetic reinforced soil integrated bridge abutment.

3.2.1 Design Considerations

As previously noted, the design of MSE wall systems often entails a shared design responsibility between material suppliers and the agency specifier/owner. The wall system supplier, who is often nationally (and not locally) based, generally designs for wall for internal and external stability with soil properties supplied by the specifier or assumed. Thus it is imperative that the specifying agency clearly define design and material definition

responsibilities, and any agency specific design or detailing requirements that vary from the primary design references (see Section 3.2.3).

3.2.2 Design Steps

The basic design steps for MSE walls are listed in Table 10-4. Some of these steps have several sub-steps in the design process. These steps are for walls with simple geometries. Steps can vary somewhat depending on type of reinforcement and/or whether or not the type of reinforcement is initially defined. Additional steps are required for more complex cases such as true bridge abutments. The design requirements are detailed in the two primary design references listed below. Detailed design guidance, discussions, example calculations, and steps for complex cases are presented in GEC 11.

Table 10-4. Basic LRFD Design Steps for MSE Walls

Step	Description
Step 1.	Establish Project Requirements – including all geometry, loading conditions (permanent, transient, seismic, etc.), performance criteria, and construction constraints.
Step 2.	Establish Project Parameters – evaluate existing topography, site subsurface conditions, reinforced wall fill properties, and retained backfill properties.
Step 3.	Estimate Wall Embedment Depth, Design Height(s), and Reinforcement Length
Step 4	Define nominal loads
Step 5	Summarize Load Combinations, Load Factors, and Resistance Factors
Step 6.	Evaluate External Stability a. Evaluate sliding b. Evaluate eccentricity c. Evaluate bearing on foundation soil d. Settlement analysis (at service limit state)

Step	Description
Step 7.	Evaluate Internal Stability a. Select type of soil reinforcement b. Define critical failure surface (for selected soil reinforcement type) c. Define unfactored loads d. Establish vertical layout of soil reinforcements e. Calculate factored horizontal stress and maximum tension at each reinforcement level. f. Calculate nominal and factored long-term tensile resistance of soil reinforcements g. Select grade (strength) of soil reinforcement and/or number of soil reinforcement elements at each level. h. Calculate nominal and factored pullout resistance of soil reinforcements, and check established layout i. Check connection resistance requirements at facing j. Estimate lateral wall movements (at service limit state) k. Check vertical movement and compression pads
Step 8.	Design of Facing Elements
Step 9.	Assess Overall Global Stability
Step 10.	Assess Compound Stability
Step 11.	Design Wall Drainage Systems. a. Subsurface drainage b. Surface drainage

Source: GEC 11

3.2.3 Primary Design References

There are two primary references for design of reinforced soil walls for transportation works. These are:

- AASHTO. (2014). *LRFD Bridge Design Specifications*. 7th Edition, with 2015 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, D.C.
- GEC 11. (2009). *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*. Authors: Berg, R.R., Christopher, B.R., and Samtani, N.C., FHWA NHI-10-024 Vol I and NHI-10-025 Vol II, Federal Highway Administration, U.S. DOT, Washington, D.C., 306p. (Vol I) and 378p. (Vol II).

Closely spaced (vertically) geosynthetic soil reinforcements in a reinforced soil wall can be designed using the references noted above or with the following references, for transportation bridges supported on a GRS-IBS system.

- FHWA. (2011a). *Geosynthetic Reinforced Soil Integrated Bridge System, Interim Implementation Guide.* Authors: Adams, M., Nicks, J., Stabile, T., Wu, J., Schlatter, W., and Hartmann, J., FHWA-HRT-11-026, Federal Highway Administration, U.S. DOT, Washington, D.C., 169p.
- FHWA. (2011b). *Geosynthetic Reinforced Soil Integrated Bridge System, Synthesis Report*. Authors: Adams, M., Nicks, J., Stabile, T., Wu, J., Schlatter, W., and Hartmann, J., FHWA-HRT-11-027, Federal Highway Administration, U.S. DOT, Washington, D.C., 64p.

MBW faced, geosynthetic reinforced walls in transportation works are normally designed with either of the two sets of references listed above. For non-transportation works, these types of walls are typically designed with the following reference. Some DOTs may allow the use of the National Concrete Masonry Association (NCMA) design method for landscape walls, but most DOTs do not allow use of this method for structural walls applications.

- NCMA. (2009). *Design Manual for Segmental Retaining Walls*. Editor: M. Bernardi, National Concrete Masonry Association, 3rd Edition, Herndon, VA, 281p.

3.3 Construction and Materials

General construction requirements and guidance for MSE walls are presented in AASHTO (2010) and FHWA (GEC 11). The general sequence of MSE wall construction is presented below. As discussed under Section 3.4, MSE walls are most often specified as a system with an approved products (wall system) list established by the specifying DOT. A system-specific and agency-specific construction erection manual should be required from each vendor applying for pre-approval of their system. This system/agency-specific erection guide should be used by the contractor, inspectors, and agency for construction.

There are a variety of soil reinforcement materials and facing materials used with MSE walls. These materials are summarized below; see GEC 11, AASHTO (2010) and AASHTO (2014) for detailed discussions and information on these materials.

3.3.1 Construction

The construction of a MSE wall entails the following general steps. More detailed guidance is material-, system-, and/or agency-specific.

- **Preparation of subgrade**. This step involves removal of unsuitable materials from the area to be occupied by the retaining structure. All organic matter, vegetation, slide debris and other unstable materials should be stripped off and the subgrade compacted. Observation and tests (e.g., proof rolling, dynamic cone penetrometer, etc.) should be performed to confirm the suitability foundation soils.
- In unstable foundation areas, ground improvement methods, such as excavation and replacement, or dynamic compaction, stone columns, etc. (see other chapters within this manual) could be used to improve the foundation constructed prior to wall structure erection.
- **Placement of a leveling pad for the erection of the facing elements**. This generally an unreinforced 6-inch thick concrete pad is often 6 inches wider, or more, than the face element. A wider concrete pad is recommended under wall curves to ensure facing does not extend over the edge of the concrete. The purpose of this pad is to serve as a guide for facing panel erection and is not intended as a structural foundation support.
- **Erection of the first row of facing on the prepared leveling pad**. Facings may consist of precast concrete panels, dry cast MBW units, large wet cast units, or other facing type. A concrete leveling pad is not used with welded wire or geosynthetic wrapped wall facings.
- With precast panels, the first row generally consists of alternating full and half-height panels. The first row of panels must be externally braced to maintain stability and alignment. Subsequent rows of panels are simply wedged and clamped to adjacent panels. Full sized blocks are used throughout the wall, with no shoring, for construction with MBW or large wet cast facings units.
- **Placement and compaction of reinforced wall fill on the subgrade to the level of the first layer of reinforcement and its compaction**. The fill should be compacted to the specified density, usually 95 to 100 percent of AASHTO T 99 maximum density and within the specified range of optimum moisture content. Compaction moisture contents within a few percent and on the dry of optimum are recommended.
- A key to good wall performance is *consistent* placement and compaction of the reinforced wall fill. Wall fill lift thickness must be controlled based on specification requirements and vertical distribution of reinforcement elements. The uniform loose lift thickness of the reinforced backfill should not exceed 12 inches. Generally an 8-inch compacted lift thickness is used, which is compatible with most facing unit heights and soil reinforcement spacing. Reinforced wall fill should be dumped into or parallel to the rear and middle of the reinforcement and bladed toward the front face.

The placement and compaction of the retained backfill lift, behind the reinforced volume, should follow the reinforced fill lift to the same elevation.

- **Placement of the first layer of reinforcing elements on the wall fill**. The reinforcements are placed and connected to the facing, when the compacted reinforced fill has been brought up to the level of or just above the connection (i.e., a gap should not exist between the fill and the connector to avoid down drag stresses on the reinforcement and/or the connector). The reinforcements are generally placed perpendicular to back of the facing panels.
- **Placement of the wall fill over the reinforcing elements to the level of the next reinforcement layer and compaction of the wall fill**. The previously outlined steps are repeated for each successive layer.
- **Construction of traffic barriers and copings**. This final construction sequence is undertaken after the final facings panel or units have been placed, and the wall fill has been completed to its final grade.

3.3.2 Wall Facings

The types of facing elements used in the different MSE systems control their aesthetics because they are the only visible parts of the completed structure. A wide range of finishes and colors can be provided in the facing, as shown in the FHWA Roadway Aesthetic Treatments Photo Showcase available at http://gallery.company39.com/FLH/gallery/. In addition, the facing provides protection against backfill sloughing and erosion, and provides, in certain cases, drainage paths. The type of facing influences the wall structure settlement tolerances. Major facing types are listed and briefly described below.

- **Segmental precast concrete panels.** The precast concrete panels have a minimum thickness of 5-½ inches, or greater, and are of a square, rectangular, cruciform, diamond, or hexagonal geometry. Typical nominal panel dimensions are 5-foot high and 5- or 10-foot wide. Precast concrete face elements can be cast in several shapes and provided with facing textures to match environmental requirements and blend aesthetically into the environment. Retaining structures using precast concrete elements as the facings can have surface finishes similar to any reinforced concrete structure. Temperature and tensile reinforcement of the concrete are required and should be designed in accordance with Section 5 of AASHTO LRFD Specifications for Highway Bridges (2014).
- **Dry cast modular block wall (MBW) units**. These are relatively small, squat concrete units that have been specifically designed and manufactured for retaining wall applications. The weight of these units ranges from 30 to 110 pounds, with units

of 75 to 110 pounds routinely used for highway projects. Unit height is typically 8 inches, though this may range from 4 to 15 inches for the various manufacturers. Exposed face length usually varies from 8 to 18 inches, with 18 inches being typical. Nominal front to back width (dimension perpendicular to the wall face) of units is typically 12 inches, but with a range of 8 to 24 inches. Units may be manufactured solid or with open cores. Full height open cores are filled with aggregate during erection. Units are normally dry-stacked (i.e. without mortar or bearing pads) and in a running bond configuration. Vertically adjacent units may be connected with shear pins, lips, or keys. etc.

- **Welded Wire Mesh (WWM)**. Wire grid can be bent up at the front of the wall to form the wall face. This type of facing may be a continuation of the soil reinforcement, such as in the Hilfiker wire faced retaining wall system. Other systems may use WWM facing units with other types of soil reinforcement. If permanent wall application, the WWM is usually galvanized. Black steel is usually used for temporary wall applications.

- **Gabion Facing**. Gabions (rock-filled wire baskets) can be used as MSE wall facing with welded wire mesh, geogrids, geotextiles or the double-twisted woven steel mesh soil reinforcements that are placed between the gabion baskets. For example, this facing system is used by The Maccaferri Terramesh® wall system uses double-twisted woven steel mesh coated with polyvinyl chloride (PVC) soil reinforcement that is integrally manufactured with the gabion baskets.

- **Geosynthetic Facing.** Geosynthetic reinforcements are looped around at the facing to form and prevent unraveling of the exposed face of the MSE wall. A smooth or a stepped facing may be formed with the wrap-around face. Geogrid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and fabric facing. These faces are susceptible to ultraviolet light (UV) degradation and, therefore, the geosynthetic should be UV stabilized. For geogrids, vegetation can grow through the grid structure and can provide both ultraviolet light protection for the geogrid and a pleasing appearance.

- **Post-construction Facing**. For either welded wire mesh or geosynthetic wrapped faced walls– whether geotextile, geogrid, or wire mesh soil reinforced – the permanent facing can be attached after construction of the wall by shotcreting, casting concrete in-place, or by attaching prefabricated facing panels made of concrete, wood, or other materials. This multi-staging facing approach adds cost but is advantageous where significant differential settlements are anticipated.

3.3.3 Soil Reinforcements

Steel and geosynthetic soil reinforcements are used with MSE walls. The common types of reinforcements, and wall facing type(s) they are typically used with, are listed below. Steel reinforcements are generally hot dip galvanized after fabrication, to provide corrosion resistance.

- Steel strips. The currently commercially available strips are: (i) ribbed top and bottom, 2 inches wide and 5/32-inch thick; and (ii) configured in the shape of a ¼-inch high sine wave, 1½, 2, or 2½ inches wide and 5/32-inch thick. Smooth strips 2- to 4¾-inches wide, 1/8 to 5/32-inch thick have been used. The steel strips are used with segmental precast concrete panel facings.
- Steel grids. Welded wire grid using two to six W7.5 to W24 longitudinal wire spaced at either 6 or 8 inches. The transverse wire may vary from W11 to W20 and are spaced based on design requirements from 6 to 24 inches. Welded steel wire mesh spaced at 2 by 2-inch of thinner wire has been used in conjunction with a welded wire facing. Reinforcements with two longitudinal wires are also described as steel ladder grids. Steel ladder grids are used with segmental precast concrete panel, MBW unit, and steel grid wall facings.
- Double twisted steel mesh. One proprietary system uses a metallic, soft-tempered, double twisted mesh soil reinforcement that is galvanized and then coated with poly vinyl chloride (PVC). This reinforcement is used in gabion faced MSE wall construction.
- High Density Polyethylene (HDPE) geogrid. These are of uniaxial manufacture and are available in a variety of strength grades. This type of reinforcement is used with segmental precast concrete panel, MBW unit, WWM, and wrap-around wall facings.
- PVC coated polyester (PET) geogrid. These are available from a number of manufacturers, and in a variety of strength grades from each manufacturer. They are characterized by bundled high tenacity PET fibers in the longitudinal load carrying direction. PET geogrid reinforcements are used with MBW unit wall, large precast block, and wrap-around facings.
- High strength geotextiles. These are available from a number of manufacturers, and in a variety of strength grades from each manufacturer. Both polyester (PET) and polypropylene (PP) geotextiles are used. Geotextile reinforcements are used with wrap-around and MBW unit wall facings.

- Geosynthetic strap. The geosynthetic strap type reinforcement is used with segmental precast concrete panel faced MSE walls. The strap consists of PET fibers encased in a polyethylene (PE) sheath.
- Geocomposites. Geogrids, knitted and woven geotextiles have been combined with nonwoven geotextiles to provide improved drainage, allowing for pore water pressure dissipation, when using poorly draining reinforced fills.

3.3.4 Reinforced Soil Fill

MSE walls require high quality wall fill for durability, good drainage, constructability, and good soil reinforcement interaction. These properties can be obtained from well graded, granular materials. All fill material used in the structure volume for MSE wall structures should be reasonably free from organic or other deleterious materials and should conform to the gradation limits listed in Table 10-5.

Note that Table 10-5 presents a broad gradation range that is applicable across the United States. Individual DOTs may adjust this range based upon locally available and economical select granular fill. The reinforced fill should be well-graded in accordance with the Unified Soil Classification System (USCS) in ASTM D2487. Furthermore the reinforced fill should conform to the shear strength angle of friction requirement, plasticity index (PI) limit, and soundness criteria as listed in Table 10-6. Reinforced fills where steel reinforcements will be used must also conform to the electrochemical properties listed in Table 10-7. Reinforced fills where geosynthetic reinforcements will be used shall conform to the electrochemical properties (i.e., pH) listed in Table 10-8.

Table 10-5. MSE Wall Select Granular Reinforced Fill Gradation Requirements

U.S. Sieve Size	Percent Passing[a]
4 inches (102 mm)[a, b]	100
No. 40 (0.425 mm)	0-60
No. 200 (0.075 mm)	0-15

Notes:
a. To apply default F* values, C_u, should be greater than or equal to 4.
b. The maximum particle size for these materials be reduced to ¾-inch for geosynthetics, and epoxy and PVC coated steel reinforcements unless construction damage assessment tests are or have been performed on the reinforcement with the specific or similarly graded large size granular fill.

Table 10-6. Additional MSE Wall Select Granular Reinforced Fill Requirments

Property	Requirement
Angle of Friction[a]: (AASHTO T 236[b])	> 34º
Plasticity Index, PI: (AASHTO T 90)	PI < 6
Soundness: (AASHTO T 104)	The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles.

Notes:

a. No testing is required for fills where 80% of sizes are greater than ¾-inch, and use of 34º.
b. On the portion finer than the No. 10 sieve, utilizing a sample of the material compacted to 95% per AASHTO T99, Methods C or D, at optimum moisture content.

Table 10-7. Electrochemical Fill Requirements for Steel Reinforcements

Property	Criteria	Test Method
Resistivity	> 3000 ohm-cm	AASHTO T 288
pH	> 5 and < 10	AASHTO T 289
Chlorides	< 100 ppm	ASTM D4327
Sulfates	< 200 ppm	ASTM D4327
Organic Content	1% max.	AASHTO T 267

Table 10-8. Electrochemical Fill Requirements for Geosynthetic Reinforcements

Base Polymer	Property	Criteria	Test Method
Polyester (PET)	pH	3 < pH < 9	AASHTO T 289
Polyolefin (PP & HDPE)	pH	pH > 3	AASHTO T 289

From a reinforcement capacity point of view, lower quality wall fills could be used for MSEW structures; however, a high quality granular wall fill has the advantages of better drainage, providing better durability for metallic reinforcement, and requiring less reinforcement. There are also significant handling, placement and compaction advantages in

using granular soils corresponding to an increased rate of wall erection and improved maintenance of wall alignment tolerances.

GRS-IBS wall fill recommendations are similar to those for MSE walls reinforced with geosynthetics, with the following differences. The minimum angle of friction is 38º. A well-graded fill may be used, but an open-graded fill is typically used. The open-graded fill is recommended for bridge abutment support applications, and required where the wall fill may be inundated. The open graded fill generally has a gradation with 100% passing the ¾-inch sieve, 0 to 5% passing the No. 50 sieve, , and consisting of crushed, angular stone. An AASHTO (M43) No. 8 and a No. 89, listed in Table 10-9 and 10-10 respectively, meet these gradation requirements. Plasticity limit and soundness requirements are as listed in Table 10-6. Electrochemical requirements are as listed in Table 10-8.

Table 10-9. AASHTO No. 8 Gradation for GRS-IBS

U.S. Sieve Size	Percent Passing
½ inches (12.5 mm)	100
3/8 inches (9.5 mm)	85-100
No. 4 ((4.75 mm)	10-30
No. 8 (2.36 mm)	0-10
No. 16 (1.18 mm)	0-5

Table 10-10. AASHTO No. 89 Gradation for GRS-IBS

U.S. Sieve Size	Percent Passing
½ inches (12.5 mm)	100
3/8 inches (9.5 mm)	90-100
No. 4 (4.75 mm)	20-55
No. 8 (2.36 mm)	5-30
No. 16 (1.18 mm)	0-10
No. 50 (0.30 mm)	0-5

3.3.5 Appurtenant Materials

Walls using precast concrete panels require bearing pads in their horizontal joints that provide some compressibility and movement between panels during elastic compression and settlement of the reinforced fill to mitigate downdrag stress on the reinforcement facing connection and preclude concrete-to-concrete contact. These materials are generally EPDM rubber or HDPE. The compressibility and thickness of the horizontal joint material should be a function of the wall height. All joints of precast concrete panels are covered along the back face with geotextile filter strips designed to prevent the migration of fines from the reinforced wall fill and allow free drainage of water through the joint. The geotextile requirements include apparent opening size (AOS) and permeability, which must be compatible with the gradation of the reinforced fill.

Bearing pads are not routinely used with MBW units. A zone of aggregate fill, usually 1-foot wide, is used behind the MBW units and within units with cores. This gravel is readily compacted and conforms to the MBW unit. A filter is required between the gravel zone and wall fill, and can either be a soil filter or a geotextile filter. Again, the filter layer must be appropriately designed (i.e. gradation of the soil filter and the AOS and permeability of the geotextile filter) to provide adequate drainage and filtration of the reinforced fill.

3.4 Overview of Construction Specifications and Quality Assurance

3.4.1 Specification Development

MSE wall systems are contracted using two different approaches:

- Performance or end-result approach using approved systems and components, with lines and grades noted on the drawings and geometric and design criteria specified. In this case, a project-specific design review and detail plan submittal occurs in conjunction with working drawing submittal.
- Agency designs with system components, drainage details, erosion measures, and construction execution explicitly specified in the contracting documents. This can be accomplished by using standard designs for common wall heights (e.g., see Minnesota DOT 2016).

Both contracting approaches are valid if properly implemented. Each approach has advantages and disadvantages. Most user agencies generally prefer the performance based specification for MSE walls. An exception to that are GRS-IBS abutments which agencies typically design and specify materials requirements of components.

3.4.2 Summary of Quality Assurance

Construction of MSE systems is relatively simple and rapid. As listed in Section 3.3, the construction sequence consists mainly of preparing the subgrade, placing and compacting backfill in normal lift operations, laying the reinforcing layer into position, and installing the facing elements. Special skills or equipment are usually not required, and locally available labor can be used, however, attention to details is essential for adequate performance and aesthetics and experienced crews can provide higher production rates. Most material suppliers provide training for construction of their systems. Training should also be required for those monitoring the construction. The outline of a checklist showing general requirements for monitoring and inspecting MSE and RSS systems is provided in Table 10-11. The table should be expanded by the agency to include detailed requirements based on the agencies specifications and the specific project plans and specification requirements.

Table 10-11. Outline of MSE Wall Field Inspection Checklist Requirements

Item	Requirements
1.	Read the specifications and become familiar with: • material requirements • construction procedures • soil compaction procedures • alignment tolerances • acceptance/rejection criteria
2.	Review the construction plans and become familiar with: • construction sequence • corrosion protection requirements • special placement to reduce damage • soil compaction restrictions • details for drainage requirements • details for utility construction • construction of slope face • contractor's documents
3.	Review material requirements and approval submittals. Review construction sequence for the reinforcement system.
4.	Check site conditions and foundation requirements. Observe: • preparation of foundations • leveling pad construction (check level and alignment) • site accessibility • limits of excavation • construction dewatering • drainage features; seeps, adjacent streams, lakes, etc.

Item	Requirements
5.	On site, check reinforcements and prefabricated units. Perform inspection of prefabricated elements (i.e. casting yard) as required. Reject precast facing elements if: • compressive strength < specification requirements • molding defects (e.g., bent molds) • honey-combing • severe cracking, chipping or spalling • color of finish variation • tolerance control • misaligned connections
6.	Check reinforcement labels to verify whether they match certification documents.
7.	Observe materials in batch of reinforcements to make sure they are the same. Observe reinforcements for flaws and nonuniformity.
8.	Obtain test samples according to specification requirements from randomly selected reinforcements.
9.	Observe construction to see that the contractor complies with specification requirements for installation.
10.	If possible, check reinforcements after aggregate or riprap placement for possible damage. This can be done either by constructing a trial installation, or by removing a small section of aggregate or riprap and observing the reinforcement after placement and compaction of the aggregate, at the beginning of the project. If damage has occurred, contact the design engineer.
11.	Check all reinforcement and prefabricated facing units against the initial approved shipment and collect additional test samples.
12.	Monitor facing alignment: • adjacent facing panel joints • precast face panels • modular block walls • wrapped face walls • line and grade

Source: GEC 11

There are some special construction considerations that the designer, construction personnel, and inspection team need to be aware of so that potential performance problems can be avoided. These considerations relate to the type of system to be constructed, to specific site conditions, the reinforced wall fill material used, and facing requirements. These items should be addressed in preconstruction reviews, prefabricated materials inspection,

construction control, and/or performance monitoring programs. See GEC 11 for detailed discussions on the quality assurance items.

3.4.3 Summary of Instrumentation Monitoring and Construction Control

Since MSE wall technologies are well established, the need for an extensive monitoring programs should be limited to projects in which new features or materials have been incorporated into the design, substantial post construction settlements are anticipated and/or construction rates require control, where degradation/corrosion rates of reinforcements are to monitored (e.g., to allow use of marginal fills), or to aid in asset management. A minimum monitoring program should be established for all structures to facilitate asset management. This could simply consist of as built drawings with a few survey points placed at the top and bottom of the structure as select locations (i.e., critical features and/or tallest portion of the wall).

If a monitoring program is to be used, the first step in planning is to define the purpose of the measurements. Every instrument on a project should be selected and placed to assist in answering a specific question. If there is no question, there should be no instrumentation. Both the questions that need to be answered and the clear purpose of the instrumentation in answering those questions should be established. The most significant parameters of interest should be selected, with care taken to identify secondary parameters that should be measured if they may influence primary parameters.

Each of the steps in the sequential construction of MSE wall systems is controlled by certain method requirements and tolerances. Construction manuals for proprietary MSE systems should be obtained from the contractor to provide guidance during construction monitoring and inspection. The construction manual should be agency-specific and not a generic manual, and should be required by an agency when they evaluate and place the wall system on an approved products list.

See GEC 11 for a detailed description of general construction steps and requirements for MSE walls. Construction controls are required with each step, to assure the quality of the constructed wall. General steps in MSE wall construction are outlined in Section 3.3.

3.5 Cost Data

Site specific costs of a soil-reinforced structure are a function of many factors, including cut-fill requirements, wall/slope size and type, in situ soil type, available wall fill and retained backfill materials, facing finish, temporary or permanent application, etc. It has been found that MSE walls with precast concrete facings are usually less expensive than reinforced concrete retaining walls for heights greater than about 10 feet and average foundation

conditions. Modular block wall (MBW) unit faced walls are competitive with concrete walls at all heights and also for small projects.

In general, the use of MSE walls results in savings on the order of 25 to 50 percent and possibly more compared with a conventional reinforced concrete retaining structure, especially when the latter is supported on a deep foundation system (poor foundation condition). A substantial savings is obtained by elimination of the deep foundations, which is usually possible because reinforced soil structures can accommodate relatively large total and differential settlements. Other cost saving features include ease of construction and speed of construction. Typical total costs for permanent transportation MSE walls range from $30 to $65 per square foot of face, and generally vary as function of height, size of project, aesthetic treatment, site accessibility, and cost of select wall fill. However, reinforced fill costs vary considerably across the United States and regional costs may be much higher than the indicated range.

3.5.1 Cost Components

The actual cost of a specific MSEW structure will depend on the cost of each of its principal components. For segmental precast concrete faced structures, typical relative costs are:

- Erection of panels and contractors profit - 20 to 30 percent of total cost.
- Reinforcing materials - 15 to 30 percent of total cost.
- Facing system - 20 to 40 percent of total cost.
- Reinforced wall fill including placement - 30 to 60 percent of total cost, where the fill is a select granular fill from an off-site borrow source.

The additional cost for panel architectural finish treatment ranges from $0.50 to $1.50 per square foot depending on the complexity of the finish. Traffic barrier costs average $170 per linear foot. In addition, consideration must be given to the cost of excavation, which may be somewhat greater than for other systems due to the required width of the reinforcement zone. MBW faced walls at heights less than 15 feet are typically less expensive than segmental panel faced walls by 10 percent or more.

4.0 REINFORCED SOIL SLOPES

4.1 Feasibility Considerations

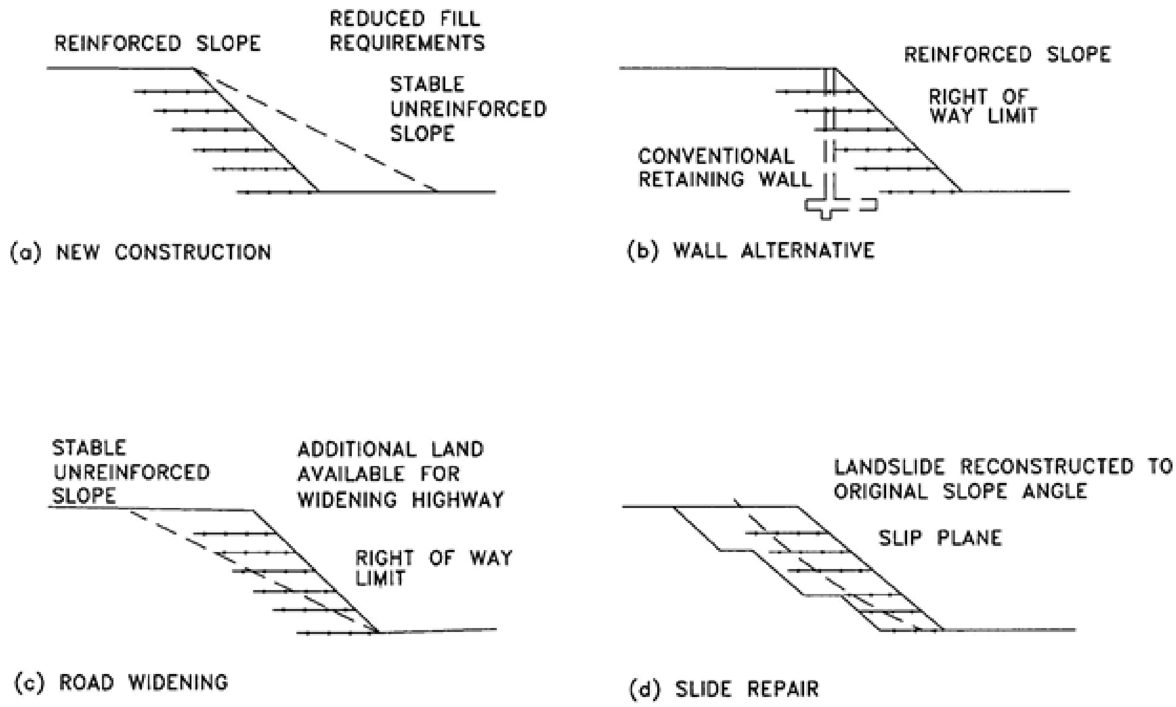
Reinforced soil slopes (RSS) are a form of mechanically stabilized earth that incorporate planar reinforcing elements (typically geosynthetics) in constructed earth sloped structures with face inclinations of less than 70 degrees. Multiple layers of reinforcement are placed in the slope during construction or reconstruction to reinforce the soil and increase the stability of the slope. RSS structures are cost-effective alternatives for new construction and reconstruction where the cost of fill, right-of-way, and other considerations may make a steeper slope desirable.

4.1.1 Applications

Reinforcement is used to construct an embankment at an angle steeper than could otherwise be safely constructed with the same soil. The increase in stability allows for construction of steepened slopes on firm foundations for new highways and as an alternative to flatter unreinforced slopes and to retaining walls. Roadways can also be widened over existing flatter slopes without encroaching beyond existing right-of-ways. RSS can also be used to repair a failed slope, usually reusing the slide debris for the reinforced fill. In this case the newly repaired slope will be safer, and reusing the slide debris rather than importing higher quality backfill may result in substantial cost savings. These applications are illustrated in Figure 10-9.

Other applications of reinforced slopes have included:

- Decreased bridge spans.
- Temporary road widening for detours.
- Prevention of surface sloughing during periods of saturation.
- Embankment construction with wet, fine-grained soils.
- Permanent levees.
- Temporary flood control structures.



GEC 11

Figure 10-9. Reinforced slope applications.

4.1.2 Advantages and Potential Disadvantages

4.1.2.1 Advantages

The economic advantages of constructing a safe, steeper RSS than would normally be possible are the result of material and right-of-way savings. It also may be possible to decrease the quality of materials required for construction. For example, in repair of landslides it is possible to reuse the slide debris rather than to import higher quality backfill. Right-of-way savings can be a substantial benefit, especially for road widening projects in urban areas where acquiring new right-of-way is expensive and, in some cases, unobtainable. RSS also provide an economical alternative to retaining walls. In some cases, reinforced slopes can be constructed at about one-half the cost of MSE wall structures.

The use of vegetated-faced reinforced soil slopes that can be landscaped to blend with natural environments may also provide an aesthetic advantage over retaining wall structures. However, there are some potential maintenance issues that must be addressed for vegetated slopes (e.g., how do you cut the grass); however, these can be satisfactorily handled in design.

In terms of performance, due to inherent conservatism in the design of RSS and greater reliability in reinforcement strength versus soil strength properties, they are actually safer

than flatter, unreinforced slopes designed at the same slope stability factor of safety. As a result, there is a lower risk of long-term stability problems developing with a reinforced slope. Such problems often occur in compacted fill slopes that have been constructed to low factors of safety and/or with marginal materials (e.g. deleterious soils such as shale, fine grained low cohesive silts, plastic soils, etc.).

2.1.2.2 Potential Disadvantages

The following general potential disadvantages may be associated with all reinforced soil structures, and are dependent upon local and project conditions:

- Require a relatively large space (e.g., excavation if in a cut) behind the slope face to install required reinforcement.
- If an end result RSS specification is used, the design of the soil-reinforced system requires a shared design responsibility between system/material suppliers and owners.

4.1.3 Feasibility Evaluations

4.1.3.1 Geotechnical

The development and implementation of an adequate subsurface investigation program for the existing foundation conditions is a key element for ensuring successful project implementation. Causes for distress experienced in projects are often traced to inadequate subsurface exploration programs that did not disclose local or significant areas of soft soils, causing significant local differential settlement. Determination of engineering properties for foundation soils should be focused on establishment of bearing resistance, global stability, settlement potential, and position of groundwater levels.

4.1.3.2 Environmental Considerations

Stability of a slope can be threatened by erosion due to surface water runoff. Erosion control and revegetation measures must, therefore, be an integral part of all reinforced slope system designs and specifications. If not otherwise protected, reinforced slopes should be vegetated after construction to prevent or minimize erosion due to rainfall and runoff on the face. Vegetation requirements will vary by geographic and climatic conditions and are, therefore, project specific. Slope face protection should not be left to the construction contractor or vendor's discretion**.** Guidance should be obtained from maintenance and regional landscaping groups in the selection of the most appropriate low maintenance vegetation.

The primary environmental condition affecting reinforcement type selection and potential performance of MSE structures is the aggressiveness of the in situ ground regime (i.e., the

reinforced fill and retained backfill and natural soils) that can cause deterioration to the reinforcement. Post construction soil environment changes must be considered where deicing salts or fertilizers are subsequently used.

Certain in situ regimes have been identified as being potentially aggressive for geosynthetic reinforcements. Polyester (PET) degrades in highly alkaline or acidic regimes (e.g., lime stabilized soils). Polyolefins appear to degrade only under certain highly acidic conditions in soils containing metals (e.g., ferric soils and mine waste). For additional specific discussions on the potential degradability of reinforcements, refer to the FHWA (2009) Corrosion/Degradation reference manual.

For steel reinforcements, in situ regimes containing chloride and sulfate salts generally in excess of 200 ppm accelerate the corrosive process as do acidic regimes characterized by a pH of less than 5 (FHWA 2009) (e.g., ferric soils). Alkaline regimes characterized by pH > 10 will cause accelerated loss of galvanization.

4.1.3.3 Site Conditions

RSS structures are particularly well suited where a "fill-type" wall must be constructed or where side-hill fills are indicated. Under these latter conditions, the volume of excavation may be small, and the general economy of this type of construction is not jeopardized. Economic advantages diminish with large cut volumes to accommodate the reinforced soil structure, but in many instances remain viable.

4.1.3.4 Aesthetics

The slope face of RSS structures is usually vegetated if approximately 1:1 or flatter. The vegetation requirements vary by geographic and climatic conditions and are therefore, project specific. Steeper slopes also may be vegetated, if a stepped facing is used to control water runoff. Slopes steeper than approximately 1:1 also may use a soil bioengineered facing or a hard armor facing (see GEC11 for more information).

4.1.4 Limitations

The design of RSS structures often assumes a stable or firm foundation. Steepening a slope significantly increases the potential for bearing capacity failure over soft soils and extensive geotechnical exploration along with rigorous analysis is required. Design charts and some design procedures do not address reinforcing the base section of a reinforced slope for construction over soft soils, which is a different type reinforcement application. The user is referred to the FHWA *Geosynthetics Design and Construction Guidelines* (FHWA 2008a)

for that application. An extension of this application is to lengthen reinforcement at the base of the embankment to improve the global stability of a reinforced soil slope.

4.1.5 Alternative Solutions

MSE walls; cantilever, gravity, semi gravity, or counterfort concrete walls; or soil embankments are the usual alternatives to an RSS structure. RSS may be cost effective in rural environments, where ROW restrictions exist or on widening projects where long sliver fills are necessary. In urban environments, they should be considered where ROW is available, as they are generally more economical than vertically faced MSE wall structures.

4.2 Construction and Materials

General construction requirements and guidance for RSSs are presented in GEC 11. The general sequence of RSS construction is presented below. As discussed under Section 4.4, RSSs may be specified as a system with vendor design and with an approved products (RSS system) list established by the specifying DOT. A system-specific and agency-specific construction erection manual should be required from each vendor applying for pre-approval of their system. This system/agency-specific erection should be used by the contractor, inspectors, and agency for construction. Alternatively, RSSs may be designed in-house by an agency, and with material specifications and construction specifications.

There are a variety of soil reinforcement materials and facing materials used with RSSs. These materials are summarized below; see GEC 11 for detailed discussions and information on these materials.

4.2.1 Construction

Construction of reinforced slopes is very similar to normal slope construction since the reinforcement layers are easily incorporated between the compacted lifts of fill. The elements of construction consist of simply:

1. Placing the soil
2. Placing the reinforcement
3. Constructing the face

The following is the usual construction sequence (GEC 11):

- Site Preparation
    - Clear and grub site.

    - Remove all slide debris (for slope reinstatement projects).
    - Prepare a level subgrade for placement of the first level of reinforcement.
    - Proof-roll subgrade at the base of the slope with a roller or rubber-tired vehicle and perform tests on the subgrade as required by the engineer.
    - Observe and approve foundation prior to fill placement.
    - Place drainage features (e.g., basedrain and/or backdrain) as required.
- Reinforcing Layer Placement
    - Reinforcement should be placed with the principal strength direction perpendicular to the face of the slope.
    - Secure reinforcement with retaining pins to prevent movement during fill placement.
    - A minimum overlap of 6 inches is recommended along the edges perpendicular to the slope for wrapped face structures. Alternatively with grid reinforcement, the edges may be clipped or tied together. When reinforcements are not required for face support, no overlap is required and edges should be butted.
- Reinforced fill Placement
    - Place fill to the required lift thickness on the reinforcement using a front end loader or dozer operating on previously placed fill or natural ground.
    - Maintain a minimum of 6 inches of fill between the reinforcement and the wheels or tracks of construction equipment.
    - Compact with a vibratory roller or plate type compactor for granular materials or a rubber-tired or smooth drum roller for cohesive materials.
    - When placing and compacting the reinforced fill material, care should be taken to avoid any deformation or movement of the reinforcement.
    - Use lightweight compaction equipment near the slope face with wrapped and welded wire mesh systems to help maintain face alignment.
- Compaction Control
    - Provide close control on the water content and density of the reinforced fill. It should be compacted to at least 95 percent of the standard AASHTO T99 maximum density within 2 percent of optimum moisture.

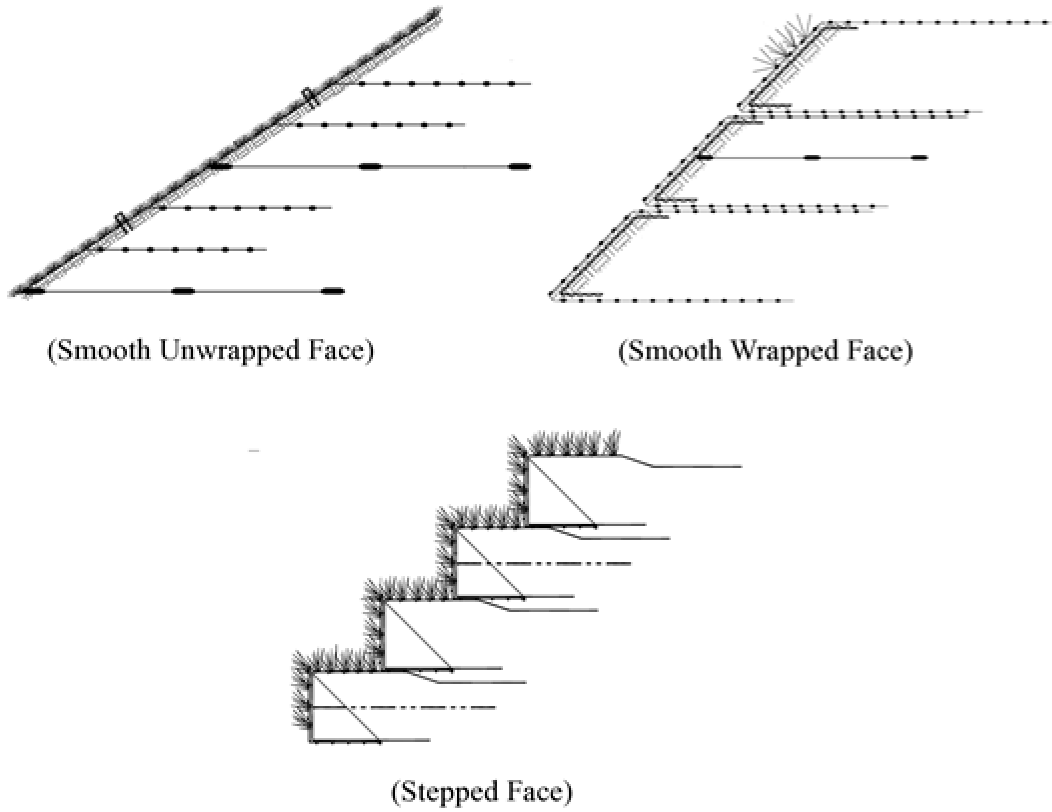
- If the reinforced fill is a coarse aggregate, then a relative density or a method type compaction specification should be used.

- Face Construction
    - Slope facing requirements will depend on soil type, slope angle and the reinforcement spacing as covered in the following section. Also see GEC 11 for additional guidance.

4.2.2 RSS Facing

If slope facing is required to prevent sloughing (i.e., slope angle is greater than soil friction strength) or erosion, several options are available. Sufficient reinforcement lengths could be provided for wrapped faced structures. A face wrap may not be required for slopes up to about 1H:1V. In this case, the reinforcements (primary and secondary) can be simply extended to the face. Compaction of the soil at the face should either be performed by extending compaction equipment over and down the slope face (e.g., using a grade all) or the fill should be extended beyond the face, compacted and then cut back to the reinforcement. For the no wrap option, a facing treatment should be applied at sufficient intervals during construction to prevent face erosion. For wrapped or no wrap construction, the reinforcement should be maintained at close spacing (i.e., every lift or every other lift but no greater than 16 inches. For armored, hard faced systems the maximum spacing generally should be no greater than 32 inches or the width of the facing.

The outward slope of an RSS is covered with a protective facing that limits erosion, protects the reinforcement and provides surficial stability. Facing types can be broken into two main categories: soft vegetated facings and hard facings. Typical RSS facings include: wrapped reinforcement facing, welded wire forms, gabions, and various concrete units. Hard facings and wrapped reinforcement typically become necessary with increased slope angle and with more erodible soils. Many facings incorporate the use of living vegetation. A live, growing face can contribute to aesthetics, limit erosion and in some cases, weave itself into the non-living reinforcement, providing additional tensile strength. Vegetation used on RSS facing are typically grassy plants and small, woody plants (in some cases bio reinforcement). Reinforcement can terminate at the slope face or be wrapped around the lift and tucked back into the slope. A wrapped face provides increased lateral confinement for the soils near the slope face. Non-wrapped slope faces are typically used for gentle slopes where less erosion and surficial sloughing are anticipated. Typical RSS face geometries, with vegetation, are shown in Figure 10-10 (Brickman and Berg 2013). Again see GEC 11 for additional guidance.



Brickman and Berg 2013

Figure 10-10. RSS face geometries – smooth, wrapped, and stepped.

4.2.3 Soil Reinforcements

Geosynthetic and steel soil reinforcements are used with RSSs. The common types of reinforcements and type of slope facing they are typically used with are listed below.

- High Density Polyethylene (HDPE) geogrid. These are of uniaxial manufacture and are available in a variety of strength grades. This type of reinforcement is used with WWM faced, wrap-around, and non-wrap slope facings.
- PVC coated polyester (PET) geogrid. These are available from a number of manufacturers, and in a variety of strength grades from each manufacturer. They are characterized by bundled high tenacity PET fibers in the longitudinal load carrying direction. This type of reinforcement is used with WWM faced, wrap-around, and non-wrap slope facings.
- High strength geotextiles. These are available from a number of manufacturers, and in a variety of strength grades from each manufacturer. Both polyester (PET) and polypropylene (PP) geotextiles are used. Geotextile reinforcements are used with

wrap-around and may be coated with shotcrete, or other material, for permanent slopes.

- Steel grids. Welded wire grid using W7.5 to W24 longitudinal wire spaced at either 6 or 8 inches. The transverse wire may vary from W11 to W20 and are spaced based on design requirements from 6 to 24 inches. Steel grids are generally used with steel grid slope facings. Welded steel wire mesh spaced at 2 by 2-inch of thinner wire has been used in conjunction with a welded wire facing.
- Double twisted steel mesh. A proprietary system that uses a metallic, soft-tempered, double twisted mesh soil reinforcement that is galvanized and then coated with poly vinyl chloride (PVC). This reinforcement is generally used with an integral double twisted steel mesh gabion face for RSS construction.

4.2.4 Reinforced Soil Fill

RSS reinforced fill requirements are not as stringent as MSE wall reinforced fills (see Section 3.3.4). Less select reinforced fill can be used for RSS since facings are typically flexible and can tolerate some distortion during construction. Even so, a high quality embankment fill meeting the following gradation requirements to facilitate compaction and minimize reinforcement requirements is recommended. All fill material used in the structure volume for RSS structures should be reasonably free from organic or other deleterious materials. The gradation guidelines listed in Table 10-12 are provided as recommended reinforced fill requirements for RSS construction. Note that Table 10-12 presents a broad gradation range that is applicable across the United States. Individual DOTs may adjust this range based upon locally available and economical select granular fill. Reinforced fills where steel reinforcements will be used must also conform to the electrochemical properties listed in Table 10-7. Reinforced fills where geosythetic reinforcements will be used shall conform to the electrochemical properties listed in Table 10-8. Additional RSS fill criteria are listed in Table 10-13.

RSS reinforced fill materials outside of these gradation and plasticity index requirements have been used successfully, as well as unsuccessfully. Issues with drainage problems, excessive distortion and settlement must be carefully evaluated with finer grained and/or more plastic soils. RSS reinforced fill compaction should be based on 95% of AASHTO T 99, and ±2% of optimum moisture, w_{opt}.

Table 10-12. RSS Reinforced Fill Gradation Requirements

U.S. Sieve Size	Percent Passing[(a)]
4 inches (102 mm)[(a,b)]	100
No. 4 (4.76 mm)	100 – 20
No. 40 (0.425 mm)	0 – 60
No. 200 (0.075 mm)	0 – 50

Notes:

a. To apply default F* values, C_u, should be greater than or equal to 4.

b. The maximum particle size for these materials should be reduced to ¾-inch for geosynthetics, and for epoxy and PVC coated steel reinforcements unless construction damage assessment tests are or have been performed on the reinforcement combination with the specific or similarly graded large size granular fill.

Table 10-13. Additional RSS Reinforced Rill Requirements

Property	Requirement
Plasticity Index, PI: (AASHTO T 90)	$PI \leq 20$
Soundness: (AASHTO T 104)	The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles.

4.3 Design Overview

There are two main purposes for using reinforcement in slopes:

- Improved stability for steepened slopes and slope repair.
- Compaction aids, for support of construction equipment and improved face stability.

The design of reinforcement for safe, steep slopes requires a rigorous analysis. The design of reinforcement for this application is critical, as failure of the reinforcement would result in failure of the slope.

The overall design requirements for reinforced slopes are similar to those for unreinforced slopes: A limit equilibrium, allowable stress approach is used and the factor of safety must be adequate for both the short-term and long-term conditions and for all possible modes of

failure. LRFD methods have not been fully developed for either unreinforced or reinforced slopes and are thus not included in this manual.

As illustrated in Figure 10-11, there are three failure modes for reinforced slopes:

- Internal, where the failure plane passes through the reinforcing elements.
- External, where the failure surface passes behind and underneath the reinforced zone.
- Compound, where the failure surface passes behind and through the reinforced soil zone.

In some cases, the calculated stability safety factor can be approximately equal in two or all three modes, if the reinforcement strengths, lengths and vertical spacing are optimized (Berg et al., 1989).

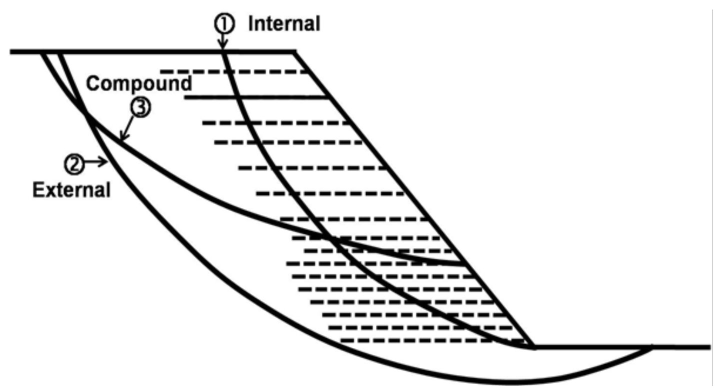


Figure 10-11. Failure modes for reinforced soil slope.

4.3.1 Design Considerations

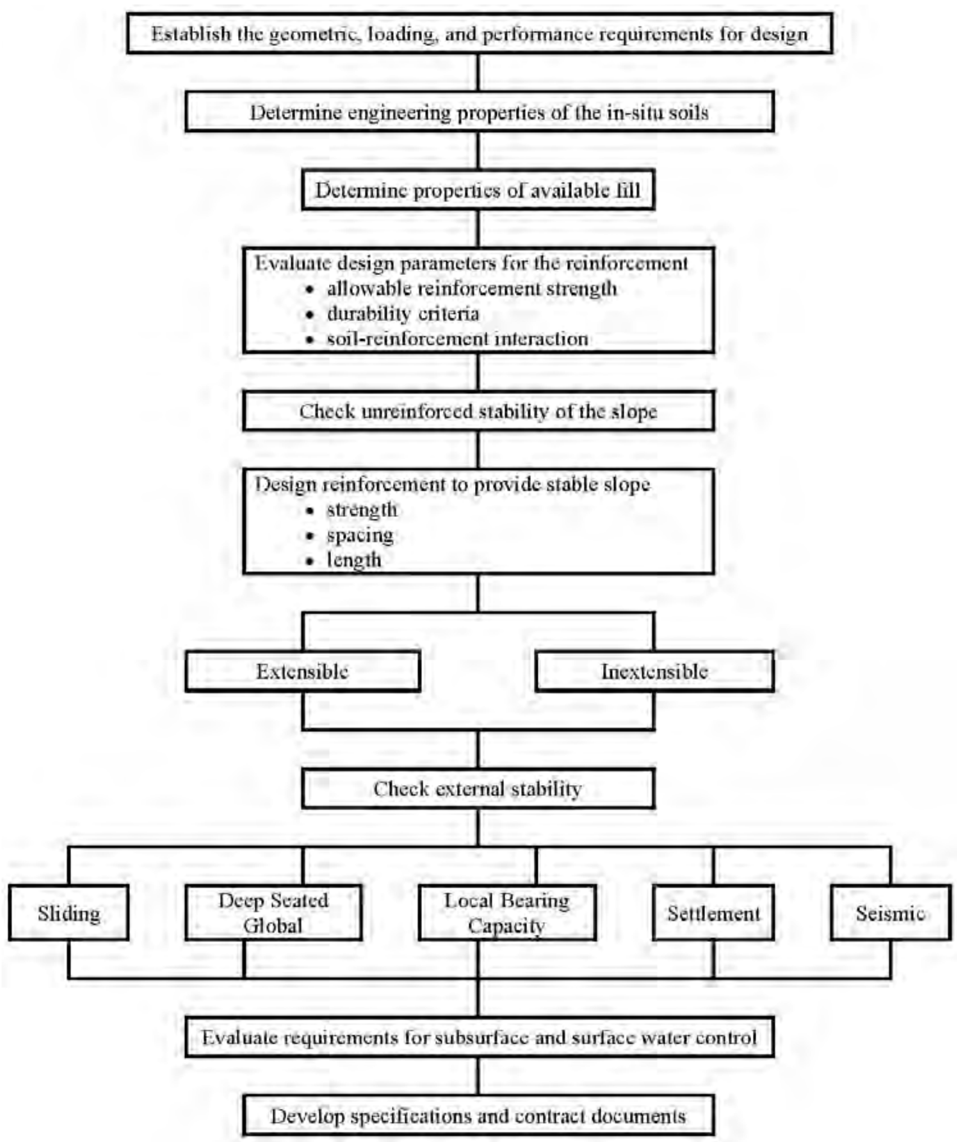
The calculations required for stability utilize conventional geotechnical design procedures, and design software, modified only for the presence of the reinforcement. Unlike a reinforced embankment on soft soils, the soil reinforcement in RSS structures carries tensile load over the life of the structure, similar to MSE walls. A key component of a RSS structure is the facing system, thus design of such is critical to the performance of the slope. Unlike MSE wall facings, a vegetated face system of a RSS requires maintenance (e.g., water, fertilizer, cutting, etc.), which must be factored into facing system selection and design.

RSS may be designed in-house by an agency or a line-and-grade approach, similar to that used for MSE walls, can be used. If an in-house approach is used, the materials (e.g., reinforcement, facing, drainage) are selected, specified and detailed by agency engineers. Stability analyses, with reasonable soil properties and stability safety factors, are performed by agency engineers. With a line-and-grade approach, the design of a RSS system entails a

shared design responsibility between material suppliers and the agency specifier/owner, similar to MSE walls as previously noted.

4.3.2 Design Steps

The basic design steps for RSS structures are listed in Figure 10-12. Some of these steps have several sub-steps in the design process. These steps are for RSSs on firm foundations. Steps can vary somewhat depending on type of reinforcement and/or whether or not the type of reinforcement is initially defined. Additional steps are required for more complex RSSs. Detailed design guidance, discussions, example calculations, and steps for complex cases are presented in GEC 11.



GEC 11

Figure 10-12. Flow chart of steps for reinforced soil slope design.

4.3.3 Primary Design References

The primary reference for design of RSS structures for transportation works is:

- GEC 11. (2009). *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes.* Authors: Berg, R.R., Christopher, B.R., and Samtani, N.C., FHWA NHI-10-024 Vol I and NHI-10-025 Vol II, Federal Highway Administration, U.S. DOT, Washington, D.C., 306p. (Vol I) and 378p. (Vol II).

Designers of RSS structures will also find the following reference very useful:

- Duncan, J.M.., and Wright, S.G. (2005). *Soil Strength and Slope Stability*. John Wiley & Sons, Inc., Hoboken, NJ, 297p.

4.4 Overview of Construction Specifications and Quality Assurance

4.4.1 Specification Development

RSS systems are contracted using two different approaches:

- Performance or end-result approach using approved systems and components, with lines and grades noted on the drawings and geometric and design criteria specified. In this case, a project-specific design review and detail plan submittal occurs in conjunction with working drawing submittal. Project specific slope face vegetation protection requirements should be defined by the specifier, and not be left to the contractor or vendor's discretion.
- Agency designs with system components, drainage details, erosion measures, and construction execution explicitly specified in the contracting documents.

Both contracting approaches are valid if properly implemented. Each approach has advantages and disadvantages. Many agencies generally prefer to design in-house and use a generic specification for the RSS components. Agencies can also use standard designs for common slope heights (e.g., see Minnesota DOT 2016).

4.4.2 Summary of Quality Assurance

Construction of RSS structures is relatively simple and rapid. The construction sequence consists mainly of preparing the subgrade, placing and compacting backfill in normal lift operations, laying the reinforcing layer into position, and installing the facing erosion protection. Special skills or equipment are usually not required, and locally available labor can be used. Most material suppliers provide training for construction of their systems. The outline of a checklist showing general requirements for monitoring and inspecting MSE and RSS systems is provided in Table 10-11. The table should be expanded by the agency to

include detailed requirements based on the agency's specifications, and on the project specific plans and specifications.

There are some special construction considerations that the designer, construction personnel, and inspection team need to be aware of so that potential performance problems can be avoided. These considerations relate to the type of system to be constructed, to specific site conditions, the backfill material used and facing erosion protection requirements. These items should be addressed in preconstruction reviews, prefabricated materials inspection, construction control, and/or performance monitoring programs. See GEC 11 for detailed discussions on the quality assurance items.

4.4.3 Summary of Instrumentation Monitoring and Construction Control

RSS technology is well established and, therefore, the need for monitoring programs should be limited to cases in which new features or materials have been incorporated into a design, substantial post construction settlements are anticipated and/or construction rates require control, or for asset management.

If a monitoring program is to be used, the first step in planning is to define the purpose of the measurements. Every instrument on a project should be selected and placed to assist in answering a specific question. *If there is no question, there should be no instrumentation.* Both the questions that need to be answered and the clear purpose of the instrumentation in answering those questions should be established. The most significant parameters of interest should be selected, with care taken to identify secondary parameters that should be measured if they may influence primary parameters.

Each of the steps in the sequential construction of a RSS is controlled by certain method requirements and tolerances. Full construction requirements and tolerances should be detailed on the project drawings and specifications for in-house agency designs. For end result specification with approved systems, the construction manuals for proprietary RSS system being used should be obtained from the contractor to provide additional (to project plans and specifications) guidance during construction monitoring and inspection. The construction manual should be agency-specific and not a generic manual, and may be required by the agency to place the RSS system on an approved products list.

See Section 4.2.1 and GEC 11 for a detailed description of general construction steps and requirements for RSSs. Construction controls are required with each step, to assure the quality of the constructed wall. The construction of RSS embankments is considerably simpler than MSE wall construction, but does consist of many of the elements outlined for MSEW construction. They are summarized as follows:

- Site preparation.
- Construct subsurface drainage features.
- Place reinforcement layer.
- Place and compact backfill on reinforcement.
- Construct face.
- Place additional reinforcement and reinforced fill.
- Construct surface drainage features.

4.5 Cost Data

The economy of using RSS must be assessed on a case-by-case basis, where use is not dictated by space constraints. For such cases, an appropriate benefit to cost ratio analysis should be conducted to determine whether a steeper slope with the reinforcement is justified economically over the alternative flatter slope with its increased right-of-way and materials costs, etc. It should be kept in mind that guardrails or traffic barriers are often necessary for steeper embankment slopes and additional costs such as erosion control systems for slope face protection must be considered.

With respect to economy, the factors to consider are as follows:

- Cut or fill earthwork quantities.
- Size of slope area.
- Average height of slope area.
- Angle of slope.
- Cost of non-select versus select backfills.
- Temporary and permanent erosion protection requirements.
- Cost and availability of right-of-way needed.
- Complicated horizontal and vertical alignment changes.
- Need for temporary excavation support systems.
- Maintenance of traffic during construction.
- Aesthetics.
- Requirements for guardrails and traffic barriers.

High RSS structures have relatively higher reinforcement and lower backfill costs. Recent (GEC 11) bid prices suggest costs ranging from \$10 to \$24/square foot as a function of height. For applications in the 30- to 50-foot height range, bid costs of about \$16/square foot have been reported. These prices do not include safety features and drainage details.

4.5.1 Cost Components

The actual bid cost of a specific RSS structure depends on the cost of each of its principal components. Based on limited data, typical relative costs are:

- Reinforcement - 45 to 65 percent of total cost
- Reinforced fill - 30 to 50 percent of total cost
- Face treatment - 5 to 10 percent of total cost

For RSS systems used as an alternate to flatter slopes, the savings in the soil alone will often pay for the reinforcement as well as providing a more sustainable solution.

5.0 SOIL NAIL WALLS

5.1 Feasibility Considerations

Soil nail walls are constructed using a top-down construction sequence, where the ground is excavated in lifts of limited height. Soil nails and an initial shotcrete facing are installed at each excavation lift to provide support. Subsequently, a final shotcrete or cast-in-place-concrete (CIP) facing is installed. Soil nail walls can be more advantageous than other top-down retaining systems where the ground can temporarily sustain short, vertical or sub-vertical unsupported cuts. Discussions about favorable and unfavorable subsurface conditions for cost-effective construction of soil nail walls that can be used to aid in evaluating their feasibility, and on the main factors affecting the construction costs of these systems is presented in GEC 7.

5.1.1 Applications

Soil nail walls can be used in the following roadway applications.

- Roadway cuts - soil nailing is attractive in roadway cuts because only a limited excavation and reasonable right-of-way (ROW) and clearing limits are required.
- Road widening under existing bridge abutments - soil nail walls can be advantageous for underpass widening when the removal of an existing bridge abutment slope is necessary. While the cost of installing a soil nail wall under a bridge abutment may be comparable to that of other options, the advantage of soil nailing is that the size of the drill rig is relatively small. Soil nailing equipment can operate within limited overhead, and traffic flow along the underpass road may not need to be totally interrupted during the widening.
- Tunnel portals - the use of soil nails in tunnel portals is similar to that for road cuts, but with additional design and construction aspects to be addressed.
- Repair and reconstruction of existing retaining structures - soil nails can be used to stabilize and/or strengthen failing or distressed retaining structures.
- Hybrid soil nail systems - soil nail walls can be used with other types of wall systems such as ground anchor walls and MSE walls to combine the advantage of each system. This situation may arise for walls with a complex layout or when the costs associated with other earth-retaining systems are too high.
- Shored Mechanically Stabilized Earth (SMSE) walls – permanent soil nail walls are combined with MSE walls, to limit excavation cuts, for widening low-volume roads in steep terrain. See Section 6.2 for a description of this related technology.

- Temporary and permanent excavation support and slope stabilization.

5.1.2 Advantages and Potential Disadvantages

5.1.2.1 Advantages

Advantages associated with soil nailing fall into three main categories: Construction, Performance, and Cost (GEC 7).

Construction advantages on a project can include:

- Soil nail walls require smaller ROWs than most other earth retention systems.
- Soil nail walls are less disruptive to traffic and cause less environmental impact compared to other construction techniques such as drilled shafts or soldier pile walls, which require relatively large equipment.
- Soil nailing causes less congestion in the excavation when compared to braced excavations.
- The installation of soil nail walls is relatively fast.
- Easy adjustments to nail inclination and location can be made when obstructions are encountered, such as boulders, piles or underground utilities.
- Soil nail wall installation is not as restricted by overhead limitation as other options. This advantage is particularly important when construction occurs under a bridge.
- Soil nailing may be more cost-effective at sites with remote access because the smaller equipment is more readily mobilized.
- Soil nails are installed using equipment that is multipurpose and can be used for other substructure elements such as underpinning or protection of adjacent, movement-sensitive structures.
- A relatively large number of qualified soil nail contractors exist.
- A widespread knowledge about soil nailing exists among engineers.
- Soil nail walls can accommodate curves and “bends” more easily than other top-down construction wall systems, which would otherwise require straight wall segments.

Performance advantages on a project can include:

- Soil nail walls are relatively flexible and can accommodate comparatively large total and differential movements.

- The measured deflections of soil nail walls are usually within tolerable limits in roadway projects when the construction is properly controlled.
- Soil nail walls have performed well during seismic events.
- Soil nail walls have more redundancy than anchored walls because the number of reinforcing elements per unit area of wall is larger.
- Sculpted facings, which can be applied to soil nail walls, provide a more natural appearance to fit in with the surrounding environment.

Cost advantages on a project can include:

- Conventional soil nail walls tend to be more economical than conventional concrete gravity walls taller than approximately 12 to 15 feet.
- Soil nail walls are typically equivalent in cost or more cost-effective than ground anchor walls when conventional soil nailing construction procedures are used.

5.1.2.2 Potential Disadvantages or Limitations

The main limitations or potential disadvantages associated with soil nailing are (GEC 7):

- In projects with strict wall movement criteria, additional measures to limit deflections may be required. These requirements would add cost. If very strict movement criteria exist, soil nails may not be a feasible option for the project.
- The existence of utilities behind the wall will likely create restrictions to the location, inclination, and length of soil nails, particularly in the upper rows of nails.
- Soil nail walls are not well-suited where large amounts of groundwater seep into the excavation.
- Permanent soil nail walls require permanent underground easements.

5.1.3 Feasibility Evaluations

5.1.3.1 Geotechnical

Soil nail walls can be used in a wide range of soil types and ground conditions. Project experience has shown that certain favorable ground conditions make soil nailing more cost-effective than other techniques. Soil nail walls can generally be constructed without complications in a mixed stratigraphy, as long as the individual layers of the soil profile consist of suitable, stable materials. Conversely, certain unfavorable soil conditions can be considered marginal or difficult for soil nailing applications and may make the use of soil nails risky and/or more costly when compared with other techniques.

Soil nailing has proven economically attractive and technically feasible in the following conditions:

- The excavated soil can stand unsupported in a 4- to 6-fot high vertical or nearly vertical cut for one to two days.
- Soil nails, when installed in a relatively permeable formation, are located above the groundwater table.
- Ground conditions allow drill holes to remain stable without using casing until the tendons are installed and the drill hole is grouted.

The following ground types are examples of conditions generally considered well-suited for soil nailing applications:

- Dense to very dense granular soils with apparent cohesion
- Weathered rock with adverse weakness planes
- Stiff to hard fine-grained soils
- Engineered fill
- Residual soils
- Glacial till

Soil conditions that are less favorable than those described above can be considered difficult or marginal. Soil nail walls have been installed in such soils successfully, but not necessarily with a consideration of being the most cost-effective option. Examples of difficult or marginal soil conditions include:

- Non-engineered fill
- Residual soils with unsuitable conditions

Soil nail walls are generally unsuitable, or are more difficult and expensive to design and construct, in unfavorable soil conditions. Unfavorable soil types and ground conditions are:

- Dry, poorly graded cohesionless soils
- Granular soils with high groundwater
- Soils with cobbles and boulders
- Soft to very soft fine-grained soils
- Collapsible soils

- Organic soils
- Highly corrosive soil or highly corrosive groundwater
- Weathered rock with unfavorable weakness planes
- Karst formations
- Loess
- Glacial till
- Expansive soils

See GEC 7 for detailed descriptions and discussions of these example favorable, less favorable, and unfavorable soil conditions.

5.1.3.2 Environmental Considerations (GEC 7)

Corrosion potential is of primary concern in soil nail applications and must be evaluated in every soil nail project. Corrosion potential is assessed through laboratory soil testing of samples obtained from field investigations and through field testing. The following properties must be assessed:

- pH (potential of hydrogen)
- Electrical resistivity
- Chloride content
- Sulfate content
- Organic content

Besides the conditions listed above, certain additional environmental and/or chemical conditions inherent in some soils make them more aggressive. Examples of aggressive soils and other factors that may increase soil corrosion potential (FHWA 2009) follows.

- Acidic soils – These soils exhibit a naturally low pH (less than 5) and include pyritic soils and soils with a high level of soluble iron, which in turn can contain acidic iron sulfides.
- Sodic soils – These are encountered in the western United States and arid environments.
- Calcareous soils – These soils are another type of alkaline soils ($7<pH<9$) that may contain large concentrations of sodium, calcium, and calcium-magnesium carbonates and sulfates. These are mildly corrosive.

- Organic soils – When soils contain organic materials, they can initiate the formation of anaerobic pockets that may become contaminated with sulfate-reducing bacteria, thereby initiating severe pitting. Potentially organic soils include peats, mucks, cinder, and bogs, all exhibiting unusually high water content, and those soils with humic acid.
- Material of Industrial Origin – Industrial fills may include slag, fly ash, or fills containing construction debris; and acid mining tailings and refuse.
- Coastal Environments – Atmospheric salts and salt laden soils in marine environments may contribute to corrosion.
- Road Deicing Salts – These are encountered in Northern States. Deicing liquid contains salts that can infiltrate into soils and contribute to corrosion potential.

5.1.3.3 Site Conditions

Site conditions to consider during feasibility evaluation are listed in Section 5.3.1 Design Consideration and in Table 10-14 Initial Design Steps and Considerations.

5.1.4 Alternative Solutions

In cut situations, where a top-down construction technique will be used, alternative wall types are ground anchored with flexible or stiff facing, sheetpile with or without deadman tiebacks, soldier pile and lagging, braced, soil mixing, and jet grouted wall structures. See FHWA (2008b) for detailed information on these alternative wall systems, and for wall type selection guidance.

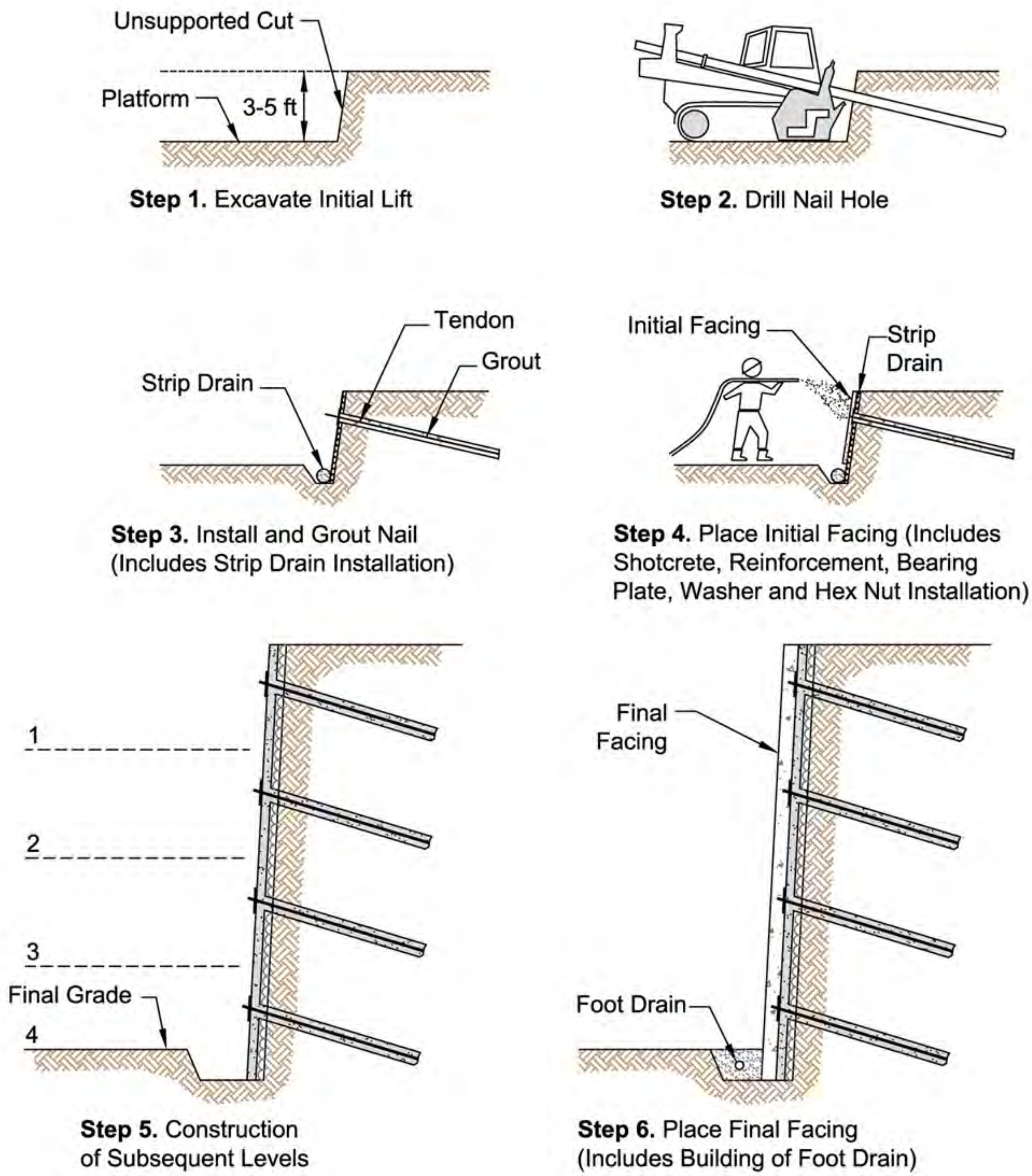
If right-of-way are soil conditions are sufficient for a temporary backcut, bottom-up type of walls may be more economical than a soil nail walls. These types of walls include cantilever, gravity, semi gravity, counterfort concrete, and MSE.

5.2 Construction and Materials

General construction requirements and guidance for soil nail walls are presented in GEC 7. The general sequence of soil nail construction is presented below. As discussed under Section 5.4, soil nail wall may be specified with a performance, end-result or a procedural, method type specification. There are a variety of materials used in construction of soil nail walls. These materials are discussed in Section 5.2.2.

5.2.1 Construction

The typical sequence of construction of a soil nail wall is described below and shown schematically in Figure 10-13 (GEC 7).



Modified after FHWA 1994

Figure 10-13. Typical soil nail wall construction sequence.

Step 1. Excavation. The depth of the initial excavation lift (unsupported cut) may range between 2.5 and 7 feet, but is typically 3 to 5 feet and reaches slightly below the elevation

where the first row of nails will be installed. The feasibility of this step is critical because the excavation face must have the ability to remain unsupported, until the nails and initial face are installed, typically one to two days. The type of soil that is excavated may limit the depth of the excavation lift. The excavated platform must be of sufficient width to provide safe access for the soil nail installation equipment.

Step 2. Drilling of Nail Holes. Drill holes are advanced using specialized drilling equipment operated from the excavated platform. The drill holes typically remain unsupported.

Step 3. **A)** Nail Installation and Grouting. Tendons are placed in the drilled hole. A tremie grout pipe is inserted in the drill hole along with the tendon; and the hole is filled with grout, placed under gravity or a nominal, low pressure. If hollow bars are used, the drilling and grouting usually take place in one operation.

B) Installation of Strip Drains. Strip drains are installed on the excavation face, continuously from the top of the excavation to slightly below the bottom of the excavation. The strip drains are placed between adjacent nails and are unrolled down to the next excavation lift.

Step 4. Construction of Initial Shotcrete Facing. Before the next lift of soil is excavated, an initial facing is applied to the unsupported cut. The initial facing typically consists of a lightly reinforced 4-inch thick shotcrete layer. The reinforcement includes welded-wire mesh (WWM), which is placed in the middle of the facing thickness. Horizontal and vertical bars are also placed around the nail heads for bending resistance. As the shotcrete starts to cure, a steel bearing plate is placed over the tendon that is protruding from the drill hole. The bearing plate is lightly pressed into the fresh shotcrete. Hex nuts and washers are then installed to engage the nail head against the bearing plate. The hex nut is wrench-tightened within 24 hours of the placement of the initial shotcrete. Testing of some of the installed nails to proof-load their capacity or to verify the load-specified criterion may be performed before proceeding with the next excavation lift. The shotcrete should attain its minimum specified 3-day compressive strength before proceeding with subsequent excavation lifts. For planning purposes, the curing period of the shotcrete should be considered 72 hours.

Step 5. Construction of Subsequent Levels. Steps 1 through 4 are repeated for the remaining excavation lifts. At each excavation lift, the strip drain is unrolled downward to the subsequent lift. A new panel of WWM is then placed overlapping at least one full mesh cell with the WWM panel above. The temporary shotcrete is continued with the previous shotcrete lift.

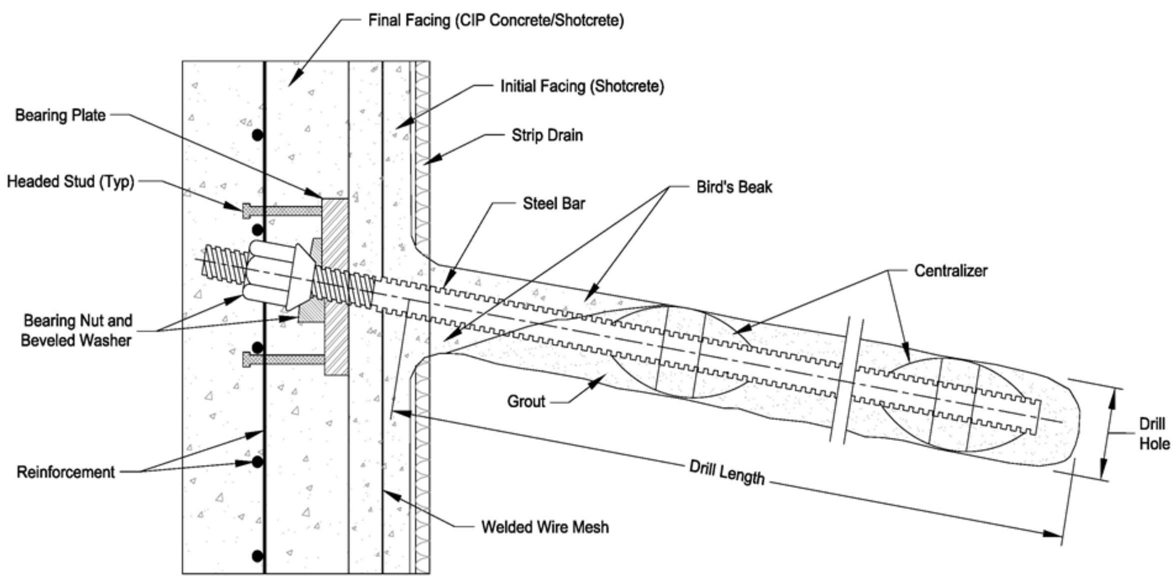
Step 6. Construction of Final Facing. After the bottom of the excavation is reached and nails are installed and tested, the final facing is constructed. Final facing may consist of CIP

reinforced concrete, reinforced shotcrete, or prefabricated panels. Weepholes, a foot drain, and drainage ditches are then installed to discharge water that may collect in the continuous strip drain or other ground water drainage systems (e.g., horizontal drain pipes).

Variations of the steps described above may be necessary to accommodate specific project conditions. For example, shotcrete may be applied at each lift immediately after excavation and before drilling of the holes and nail installation, particularly where stability of the excavation face is a concern. Another variation may be grouting the drill hole before placement of the tendon in the wet grout.

5.2.2 Materials

The main components of soil nail walls, with a solid bar tendon, are illustrated in Figure 10-14. These materials are discussed below. See GEC 7 for detailed discussions and information on these materials and components, and for guidance on use of hollow bar soil nail (HBSN) tendons.



Modified after FHWA 1994

Figure 10-14. Main components of a solid bar soil nail and wall facing.

5.2.2.1 Tendons

Solid bar soil nails are the most commonly used soil nails. They are readily available in many tendon sizes, thread types, and steel grades, and with a variety of corrosion protection schemes to suit a variety of applications and site conditions. Solid bar soil nails are placed in typically 4- to 8-inch diameter drill holes that are drilled and grouted in a two-step operation. In the first step, drill holes are drilled at a shallow angle (usually 15 degrees from horizontal)

using cased or open-hole techniques. In the second step, the tendons are inserted and grouted in the drill holes. Tendons generally have a nominal tensile strength of 60 ksi (Grade 60) or 75 ksi (Grade 75). Tendons used for soil nails are threaded. Solid bar soil nails commonly include tendons with size designations Nos. 8, 9, 10, and 11. The maximum manufactured length of threaded bars is 60 feet. Tendons are generally continuous without splices or welds. (GEC 7)

5.2.2.2 Connection Components

The steel components that connect soil nails to the facing consist of bearing plates, beveled washers, hexagonal nuts, washers and headed studs. The bearing plate, hex nuts, and washers provide connection between the nail and the initial facing, while the headed studs connect the nail end and the final facing. The purpose of the bearing plate is to distribute the force applied at the nail end onto the initial shotcrete facing and the soil behind the facing. The bearing plate is commonly Grade 36 (AASHTO M183/ASTM A36) or Grade 50 steel, and is usually square and flat, with 8- to 10-inch side dimensions and typical thicknesses of 0.75- to 1-inch. (GEC 7)

5.2.2.3 Grout

Grout is used to fill the annular space between the tendon and the soil in the drill hole. Generally, it is a neat cement grout. Grout is pumped shortly after the solid bar tendon is placed in the drill hole to reduce the potential for squeezing or caving of the hole. The grout is injected by tremie methods through a grout tube. Due to the fluid nature of the grout and the inclination of the drill hole, the fresh grout cannot fill the space above the bottom elevation of the drill hole opening. This space, called a “bird’s beak” due to its shape, is commonly filled with shotcrete either by hand-packing or during the shotcrete facing placement. (GEC 7)

5.2.2.4 Centralizers

Centralizers are installed at regular intervals along the length of each solid bar to ensure that a minimum thickness of grout completely covers the tendon. Centralizers are securely attached to the solid bar tendons and are generally polyvinyl chloride (PVC) or other non-corrosive synthetic materials. (GEC 7)

5.2.2.5 Corrosion Protection Elements

Other devices, in addition to the cement grout, are used to provide additional corrosion protection, as necessary. The solid bar tendon can be protected by encapsulation in a sheath of corrugated high-density polyethylene (HDPE) or of corrugated PVC tubing. Sheathed and

grouted bars are pre-manufactured by tendon suppliers. Once the encapsulated bar is placed in the drill hole, the annulus between the sheath and the drill hole is grouted using tremie methods.

Additional corrosion protection can be provided by coating the steel tendon. A fusion-bonded epoxy coating, which is a dielectric material that impedes the flow of electric currents, can be applied to solid bars. Solid tendons and their hardware can also be hot-dip galvanized, which provides a sacrificial material for corrosion protection. (after GEC 7)

5.2.2.6 Wall Facings

Permanent soil nail wall facings commonly consist of an initial facing and a final facing. The initial facing commonly consists of shotcrete, welded wire mesh reinforcement, and short reinforcement waler bars and vertical bars around the nail heads. The purpose of the initial facing is to support the exposed soil between the nails during excavation and nail installation, provide initial connection among nails, and furnish protection against erosion and sloughing of the excavation face. The final facing is commonly constructed of cast-in-place reinforced concrete or reinforced shotcrete. The final facing is used to meet long-term structural design resistance and to provide an aesthetic finish to the wall structure. See GEC 7 for a discussion and illustration of plain shotcrete, sculpted shotcrete, cast-in-place concrete, and precast concrete panel facing systems.

5.2.2.7 Drainage

Geocomposite strip drains (referred herein as strip drains) are installed vertically behind the initial facing along the excavation face to eliminate or minimize the development water pressure on the wall face. Geocomposite strip drains consist of a drainage core and a filtration geotextile attached to or encapsulating the core. Fitting are used with the strip drains to discharge the into a pipe drain and/or through weepholes.

It should be noted that the geocomposite strip drains will likely not eliminate hydrostatic pressure from groundwater, as the water flow will be intercepted by these drains at some level above the base of the wall. Therefore if the ground water level is above the base of the wall, the soil nail wall must either be designed for hydrostatic pressure or a dewatering drainage system must be installed (e.g., horizontal drains). For high ground water levels, a flow net should be developed for the soil nail wall to fully evaluate drainage requirements.

5.3 Design Overview

The soil nail wall design presented in GEC 7 relies on ASD- and limit equilibrium-based slope stability calculations to quantify nominal soil nail and component loads and

corresponding slip surface geometries. Those nominal loads are then used to perform LRFD checks. The LRFD framework contained in GEC 7 considers service, strength, and extreme-event limit states, consistent with those of AASHTO (2014). Recommended resistance factors for soil nail walls are presented in GEC 7, which are to be used with load factors presented in AASHTO (2014).

5.3.1 Design Considerations

Engineers must become familiar with project-specific requirements of the proposed work that would affect the design, and/or the construction and performance, of a soil nail wall. These project-specific requirements include the following:

- Preliminary site development plans indicating height, length and location of the wall and related to new infrastructure
- Physical constraints (wall near a bridge abutment, wall constructed as a cut in steep terrain, wall near a waterway or subject to scour)
- Potential effects from wall construction and use on existing and/or future, adjacent structures that may be sensitive to wall movement (bridge abutment, building, etc.)
- Accessibility to and ROW at project site
- Overhead and lateral limitations
- Presence of existing or new utilities in front, under, and behind the proposed walls
- Aesthetics of the wall finish
- Need for partial and/or full traffic closure during construction
- Availability of staging areas during construction

While the above requirements are not unique to soil nail walls, they must be carefully considered during the initial stage of their design. A field reconnaissance is highly recommended to help ascertain some of the above-listed conditions. GEC 7

5.3.2 Design Steps

The major steps and sub-steps for the initial design considerations and for design of soil nail walls are sequentially listed Table 10-14 and 10-15, respectively. The design requirements are detailed in the primary design reference listed below. Additionally, detailed design guidance, discussions, and example calculations are presented in that reference.

Table 10-14. Initial Soil Nail Wall Design Steps and Considerations

Step	Description
Step 1	Project Requirements • Establish project requirements, standards and constraints • Establish project performance • Assemble preliminary geotechnical information
Step 2	Subsurface Exploration and Development of Parameters for Design • Plan and conduct subsurface exploration • Conduct soil laboratory testing program • Establish soil corrosion potential and level of corrosion protection • Develop subsurface profiles for analysis • Develop soil parameters for design • Obtain seismic parameters • Conduct a risk analysis
Step 3	Load Definition • Define unfactored service loads • Select load combinations and load factors

Source: GEC 7

Table 10-15. Main Soil Nail Wall Design Steps

Step	Description
Step 4	Soil-Nail Configuration and Material Selection • Develop wall layout • Develop soil nail cross sections • Select soil nail pattern on wall face • Evaluate soil nail horizontal splaying • Detail corrosion protection • Select soil nail type and material properties
Step 5	Selection of Resistance Factors
Step 6	Overall Stability Analyses • Evaluate internal stability • Evaluate global stability • Evaluate basal heave (if applicable) • Evaluate sliding stability (if applicable)

Step	Description
Step 7	Strength Limit States (Geotechnical and Structural) • Verify pullout resistance • Verify sliding stability (if applicable) • Verify nail tensile resistance • Verify facing bending/flexural resistance • Verify facing punching shear resistance • Verify facing headed stud resistance • Other facing design considerations
Step 8	Service Limit States (Deformations) • Evaluate wall lateral and vertical displacements • Evaluate lateral squeeze (if applicable)
Step 9	Seismic Design • Select design seismic parameters • Adjustment of design seismic coefficients • Evaluate overall stability with seismic loads
Step 10	Drainage and Drainage Details • Evaluate internal drainage • Evaluate surface water runoff • Develop drainage details • Specialty items (if present)
Step 11	Other Design Considerations • Develop final constructability evaluation • Prepare plan for load-testing program • Prepare plan for geotechnical monitoring program
Step 12	Preparation of Construction Drawings and Specifications

Source: GEC 7

5.3.3 Primary Design References

The primary reference for design of soil nail walls for transportation works is:

- GEC 7. (2015). *Soil Nail Walls Reference Manual*. Authors: Lazarte, C.A., Robinson, H., Gómez, J.E., Baxter, A., Cadden, A., and Berg, R.R., FHWA NHI-14-007, Federal Highway Administration, U.S. DOT, Washington, D.C., 425p.

The current AASHTO (2014) specifications do not address soil nail walls. However, it is anticipated that soil nail walls will soon be included in future updates of this specification.

The installation and construction procedures are an integral part of the design process. The above reference contains detailed discussions on construction quality assurance. Greater detail on inspection methods, nail testing, inspection forms, and addressing difficult ground condition encountered during construction are provided in:

- FHWA. (1994). *Soil Nailing Field Inspectors Manual*. Authors: Porterfield, J.A., Cotton, D.M., and Byrne, R.J., Demonstration Project 103, FHWA-SA-93-068, Federal Highway Administration, U.S. DOT, Washington, D.C., 104p.

5.4 Overview of Construction Specifications and Quality Assurance

5.4.1 Specification Development

Two types of contracting methods can be used to develop contract drawings and specifications for permanent soil nail wall systems:

- Procedural or method approach with all details of design, construction materials, and methods specified in the contract documents.
- Performance or end-result approach, where the contract documents specify the criteria for the successful outcome or result of the soil nail wall. A full design and detailing of the soil nail wall, and construction methods, are not prescribed. With this approach, a detailed design submittal occurs in conjunction with the submittal of construction drawings.

Both contracting approaches are valid if properly implemented, and each has advantages and potential disadvantages. The contracting method is selected based on: the criticality and complexity of the project, the experience of the Owner and their engineering consultants, and the availability of specialty contractors. See GEC 7 and/or *GeoTechTools* for detailed specification guidance and example specifications.

5.4.2 Summary of Quality Assurance

Construction quality assurance of soil nail walls includes the primary activities of:

- Inspection of construction materials
- Inspection of construction activities
- Identification of site or soil conditions that require modification of installation procedures and/or wall design
- Load testing of installed nails

Construction material inspection includes verification of material quality, storage of materials, and inspection of the corrosion protection components and system. Primary soil materials are steel components, cement, water, and corrosion protection components. Steel components such as tendons, bearing plates, nuts, washers, welded wire mesh (WWM), and reinforcing bars are normally accepted based on mill certifications. In addition to the manufacturer's certifications, visual inspection of the materials is necessary to establish whether they have suffered damage during transport or storage. Cement is accepted based on certification by the cement producer, pending the results of unconfined compressive testing on grout and observation of soil nail installation. Water used in the preparation of grout or shotcrete must be free from chemical or organic material that may negatively affect its performance. Potable water is generally suitable for preparation of grout or shotcrete without the need for testing. The corrosion protection systems of all tendons must be inspected at the project site, prior to acceptance and use. Tendons with damaged corrosion protection must be repaired, if possible, or replaced.

The major steps in construction of soil nail walls and inspection issues for each are Table 10-16. See FHWA-NHI-14-07 (GEC 7) for discussion of these inspection items. Note that FHWA (1994) provides considerably greater detail on inspection methods, nail testing, inspection forms, and the handling of difficult ground conditions during construction.

Table 10-16. Inspector Responsibilities for a Typical Soil Nail Wall

Construction Phase	Items
Contractor Set Up	• Review Plans and Specifications • Review Contractor's schedule • Discuss anticipated ground conditions and potential problems with Contractor • Review Contractor's methods for surface water control and verify adequacy throughout construction • Review corrosion protection requirements from the Specifications and confirm that Contractor is following these requirements • If specified, obtain test samples from steel components, centralizers, and drainage materials and check all Mill test certificates for compliance with Specifications
Nail Storage and Handling	• Nails, cement, and bars must be kept dry and stored in a protected location • Nails and bars should be placed on supports to prevent contact with the ground

Construction Phase	Items
Excavation	• Prior to starting excavation, check for any variance between actual ground surface along the wall line and that shown on the Plans • Collect excavated soil samples and perform visual identification. Inform Engineer of the results for comparison against the assumed soil type for design. • Confirm that stability of excavated face is maintained at all stages of construction • Confirm that excavations are constructed within Specification tolerances of the design line and grade • For each excavation lift, confirm that Contractor is not over excavating • Enforce specific excavation sequencing plan provided on the Plans as they relate to lift thickness, length of open unsupported excavation, and, if required use of stabilizing berms • Identify areas of excessive seepage and report to Engineer • Confirm that excavated face profile is sufficiently smooth to facilitate shotcrete placement and to minimize overages in shotcrete quantities
Drilling of Nail Holes	• Confirm that drilling technique used is consistent with ground conditions • Document drilling procedures and report to the Engineer if drilling method unsuitable for actual ground conditions encountered • Confirm that soil nail hole is drilled within acceptable tolerances of the specified location, length, and minimum diameter • Observe and document locations of excessively hard drilling • Visually inspect for loss of ground or drill hole interconnection and confirm that neither of them are occurring during drilling; subsidence of ground above drilling location or large quantities of soil removal with little or no advancement of the drill head should not be permitted

Construction Phase	Items
Tendon Installation and Grouting	• Inspect open soil holes for caving or loose cuttings using a high intensity light • Inspect all soil nail bars and reinforcing steel for damage and defects prior to installation • Confirm consistency of epoxy coated or encapsulated tendons and inspect for any damage to corrosion protection prior to installation into drill hole • Confirm mix design compliance of soil nail grout and take grout samples as required • Record volume of grout placed for each drill hole • Confirm that nail bars are inserted to the minimum specified length • Confirm that centralizers are installed at specified intervals • Confirm that all required hardware is appropriately affixed at the soil nail head • Confirm that no damage occurs to corrosion protection components during installation • Confirm that grout is injected by tremie pipe starting at the bottom of the hole and that the end of tremie pipe always remains below the level of the grout as it is extracted • Confirm that grout is continued to be pumped as the grout tube, auger, or casing is removed • Confirm that the Contractor does not reverse the auger rotation while grouting except as necessary to initially release the tendon • Confirm that grout is batched in accordance with approved mix designs • Observe Contractor’s methods to place grout/shotcrete just behind the soil nail head and confirm continuous coverage • Confirm that any required testing for grout strength is conducted in accordance with specified testing methods

Construction Phase	Items
Load Testing	• Obtain all required calibration certifications of Contractor's load testing equipment • Check all deformation gauges and confirm movements during load testing • Confirm that load testing of individual nails does not commence until minimum grout curing time has passed • Confirm that the load test is performed consistently with Specifications and all required load test data is provided to permit comparison to acceptance criteria outlined in Specifications • If the soil nail fails, report to the Engineer and do not allow any retesting until the Contractor modifies the installation procedures
Drainage Installation	• Confirm compliance of drainage materials with Specifications • Confirm that geocomposite drain strips and weep hole outlet pipes are installed as specified and Plans and that the drain elements are sufficiently interconnected and provide continuous drainage paths
Wall Facing	• Confirm shotcrete mix design consistent with Specifications • Confirm that steel reinforcing is appropriately positioned within temporary shotcrete facing • Confirm that exposed soil face is covered with shotcrete within specified time limits • Confirm that minimum shotcrete thickness is maintained at all sections of the work • Confirm that shotcrete installation methods used in the field are consistent with the Specifications and as approved by the Engineer • Confirm that construction joints are clean and acceptable for shotcrete placement • Confirm that shotcrete is batched in accordance with the approved mix design • Confirm that wall finish line and grade is in accordance with Plans and Specifications • If specified, confirm that shotcrete test panels are prepared, cured, and transported to the Testing Laboratory
Post Installation	• Verify pay quantities

Source: FHWA 2008b

The contractor and the inspector both should understand the soil type and conditions that the design and planned construction techniques are based upon. It is important that the quality

assurance process identifies soil conditions outside of those planned for. Such conditions may require modification to the construction installation procedures and/or wall design

Load testing of individual soil nails consists of verification tests and proof tests. Verification tests are performed to verify the pullout resistance resulting from the contractor's installation methods with the values of pullout resistance and bond strengths used in design. Proof tests are conducted during construction, generally on a minimum of 5 percent of the total production nails, and are intended to verify that there are no significant variations in soil nail performance throughout the wall. The inspector should observe and log the results of the load tests and should ensure that the load testing schedule follows the specifications and the approved, contractor testing submittals. The inspector may also provide input during selection of proof test nails based on construction observations.

5.4.3 Summary of Instrumentation Monitoring and Construction Control

Performance monitoring instrumentation for soil nail walls could include inclinometers, wall survey points, load cells, and/or strain gauges. Inclinometers and survey points are used to measure wall movements during and after construction. The development and distribution of the nail forces may be measured with strain gauges to provide information to improve future designs. Performance monitoring should be included in any critical or unusual soil nail wall installation. Monitoring for a period of at least 1 year after construction is recommended to examine service deformation and stress development in the nails and wall facing as a function of load, time, and environmental changes such as winter freeze-thaw cycles. (GEC 7)

If a monitoring program is to be used, the first step in planning is to define the purpose of the measurements. Every instrument on a project should be selected and placed to assist in answering a specific question. *If there is no question, there should be no instrumentation*. Both the questions that need to be answered and the clear purpose of the instrumentation in answering those questions should be established. The most significant parameters of interest should be selected, with care taken to identify secondary parameters that should be measured if they may influence primary parameters.

5.5 Cost Data

The construction costs of permanent soil nail walls in public transportation projects typically range from approximately $70 to $100 per square foot of wall in 2014. When compared to typical cost ranges for other commonly used retaining structures on United States' highway projects, soil nail walls can provide a 10 to 30 percent cost savings, when conventional soil

nailing construction procedures are used. However, the actual construction cost may be considerably lower or higher depending on the factors listed below. (GEC 7)

5.5.1 Factors Affecting Cost and Construction Schedule

The following list provides some important factors that must be considered in cost comparison for soil nail walls. See GEC 7 for a more detailed list and discussions.

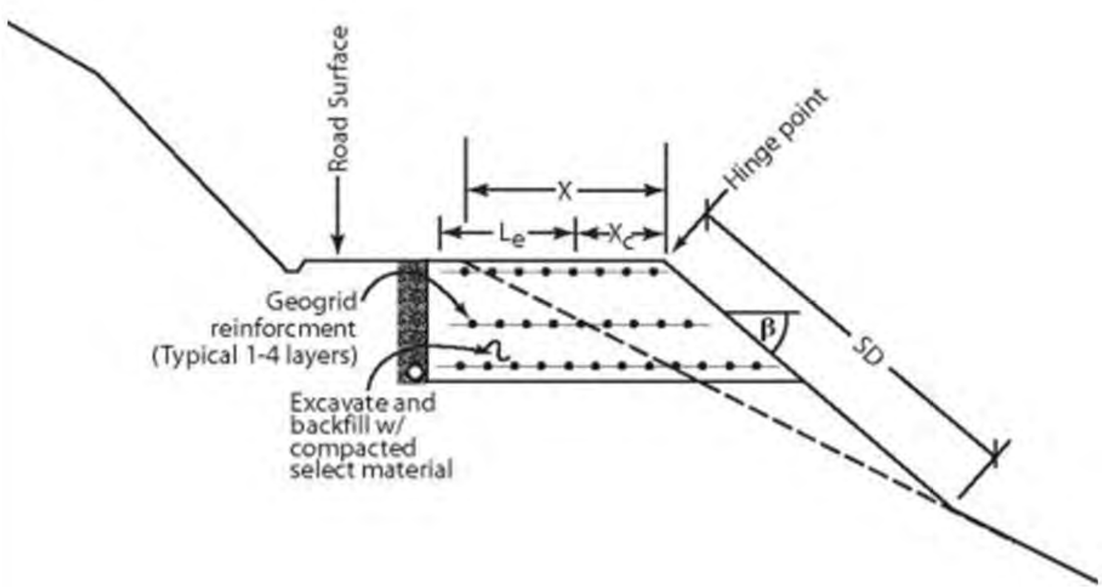
- Availability of specialty contractors skilled in soil nailing and shotcrete application near the project site
- Cost of mobilization to the site
- Site accessibility and right-of-way
- Sufficient space to operate equipment including ability to have a drill platform
- Large variations of ground conditions over small distances leading to frequent changes of drilling equipment and procedures
- Need for temporary support in exposed soil between excavation lifts, including placement of an intermediate soil berm
- Difficult conditions for advancing drill holes as a result of obstructions (e.g., utilities, large aggregate or stone particles, etc.)
- Need to provide in-hole soil stability such as casing to avoid drill hole collapse during drilling
- Special requirements for facing aesthetics, including the use of precast concrete panels or sculpted facings
- Regional conditions including high seismicity and frost susceptibility
- Changing ground conditions requiring additional numerous verification load tests

6.0 RELATED AND ALTERNATIVE TECHNOLOGIES

6.1 Deep Patching

The deep patch is a temporary mitigation technique for sliding roadway sections in steep terrains. It is typically used on roads that suffer from chronic slide movements that are primarily the result of side cast fill construction. One of the main advantages of the deep patch technique is that it is constructed with equipment that operates from the roadway and does not require accessing the toe of the failed area. This technique is generally not expected to completely arrest movement seen in the road but rather slow it down to manageable levels.

Deep patch repairs consist of reinforcing the top of a failing embankment with several layers of soil reinforcement, as illustrated in Figure 10-15. This work is typically performed with a small construction crew consisting of a laborer, hydraulic excavator, and a dump truck. The design is based on determining the extent of the roadway failure based on visual observations of cracking and then using analytical or empirical methods for determining the reinforcement requirements. An empirical design procedure is presented in Highway Deep Patch Road Embankment Repair Application Guide which was produced by the U.S. Department of Agriculture (USDA) Forest Service in partnership with FHWA Federal Lands Highway Division 9 (Musser and Denning 2005) and a review of the design methodology is presented in FHWA (2012).

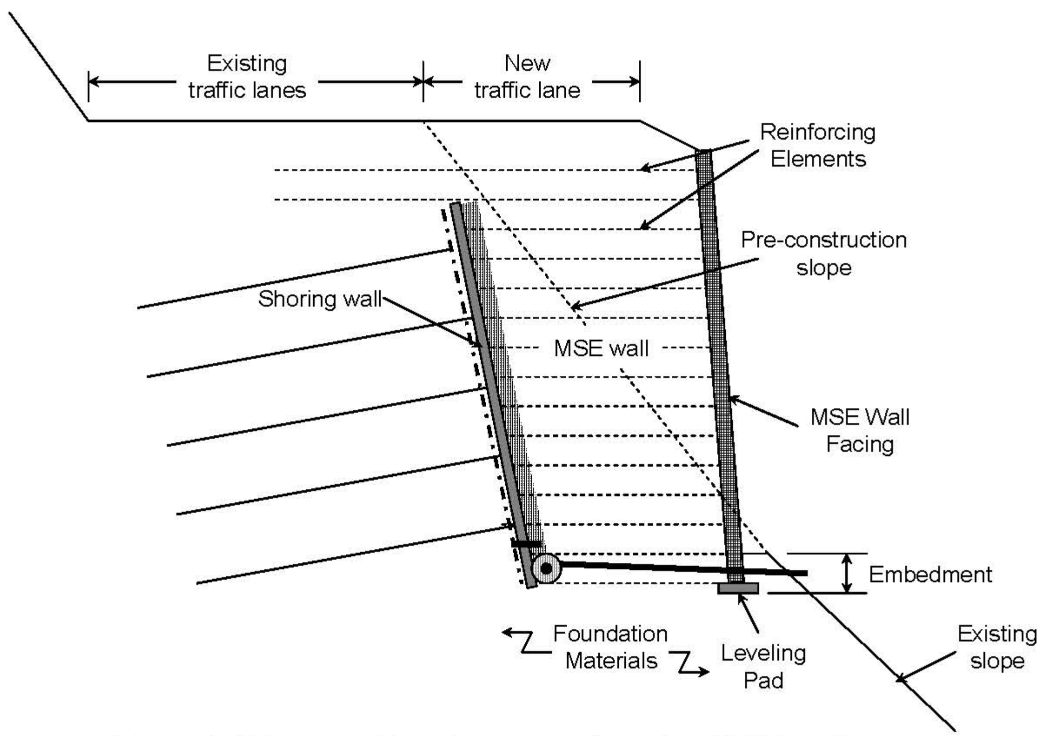


Musser and Denning 2005

Figure 10-15. Deep patch roadway embankment repair.

6.2 Shored MSE Walls for Steep Terrains and Low Volume Roads

In steep terrains MSE wall construction necessitate excavation to establish a flat bench to accommodate the soil reinforcements with a minimum length of greater than 8 feet or 70% of the height of the wall. Additionally, the required depths of embedment are proportional to the steepness of the slope below the wall toe. In some cases, the excavation required for construction of a MSE wall becomes substantial, and unshored excavation for the MSE wall is not practical, particularly if traffic must be maintained during construction of the MSE wall. Shoring, most often in the form of soil nail walls, has been employed to stabilize the backslope (or back-cut), with a MSE walls being designed and constructed in front of it. A generic cross-section of this configuration is shown in Figure 10-16. In this configuration, if the shoring wall is designed as a permanent wall it can significantly reduce the long-term lateral pressures on the MSE wall. Such MSE wall configuration is known as a shored MSE or SMSE wall. Details of SMSE walls systems are presented in FHWA (2006). Additional design and detailing guidance is provided in GEC 11.

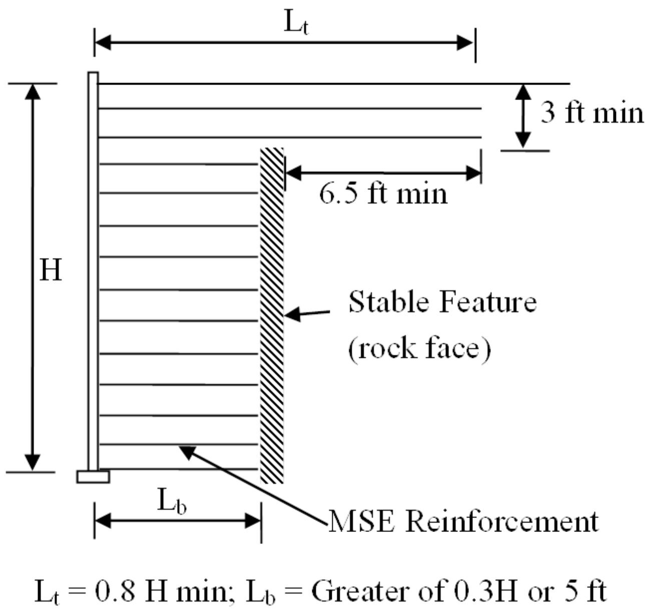


FHWA 2006

Figure 10-16. Generic cross-section of a shored MSE (SMSE) wall system for steep terrains.

6.3 Stable Feature (SFMSE) MSE Walls

MSE walls can be considered in front of apparently stable features such as a rock face as shown in Figure 10-17. Depending on the space between the MSE wall face and the stable feature, the behavior of the SFMSE wall may be similar to that of a SMSE wall. Guidelines for designing and detailing for such cases are summarized in GEC 11.



GEC 11

Figure 10-17. Minimum recommended geometry of a stable feature MSE (SFMSE) wall system.

6.4 Shoot-In Nails

Shoot-in nails use a high pressure system to insert passive inclusions into the ground to construct a temporary soil nail wall. The bars launched into the soil at high speeds over 200 miles per hour at pressures approaching 2500 psi. Bars can be perforated fiberglass, perforated galvanized steel tube, or bare steel tubes. Bars are typically 1½ inches in diameter and up to 20 feet in length. An epoxy-coated, small-diameter threaded bar can be inserted into the tube after pressure grouting is applied to increase structural capacities.

Shoot-in nails allow for a fast installation with little impact to the project site; however, it may be difficult to control the length of nail that penetrates the ground. Advantages include rapid construction, easy monitoring and testing, construction with limited headroom and right-of-way, and ability to withstand large deformations. Potential disadvantages with shoot-in nails include: (i) this is a proprietary and licensed technology; (ii) specialized contractor and equipment are required; and (iii) lack of simple, comprehensive design procedures.

This technique is applicable to landslide repairs, and to roadway and embankment widening. The bars are generally not drilled and grouted and, thus, this technology does not meet the definition of a soil nail presented in GEC 7. A shoot-in nail acts as a dowel in the soil, and the contribution to stability is primarily by shear and associated, localized bending, and not primarily by tension as with a drilled and grouted nail. Shoot-in nails develop limited axial capacity without grout. However, the technique mentioned above includes inserting a threaded bar for some to allow pressure grouting, and some increased capacity is possible. A shoot-in nail installation is shown in Figure 10-18. Detailed information on this technology is available at *GeoTechTools*, in USDA (1994a and 1994b), and in Malouf and Collin (2013).

Photograph courtesy of GeoStabilization International

Figure 10-18. Shoot-in nail installation.

6.5 Screw-In Nails

Screw-in soil nailing consists of helical soil nails that stabilize retained soils. These nails typically comprise a 1.5-inch square solid steel shaft on which steel helices are welded at regular intervals. Helical soil nails are installed using drilling equipment with sufficient torque output to penetrate the native soils. The spacing of the helices is a function of the helix diameter and is typically about 3.6 times the diameter. Screw-in nails are typically used in places difficult to access or for small areas (Collin and Cowell 1998). The bars are not drilled and grouted and, thus, this technology does not meet the definition of a soil nail, as defined in GEC 7. A screw-in nail acts as an anchor in the soil, and the contribution to stability is by bearing resistance of the helices, not by bond stresses developed along the reinforcement as

is the case with a drilled and grouted soil nail. The installation of a screw-in nail is shown is Figure 10-19. The major concern in using screw-in nails is corrosion, as galvanization and/or coatings will be damaged during installation creating more exposure to the environment. Detailed information on this technology is available at the *GeoTechTools*. (GEC 7)

Photograph courtesy of Hubbell Power Systems, Inc.

Figure 10-19. Screw-in nail installation.

6.6 Construction Working Platform

Construction working platforms use geosynthetic reinforcements in granular fill to form a temporary construction platform to support construction equipment and traffic over soft soils in order to avoid the formation of mud waves and excessive ruts. The contribution of the geosynthetic layer is to increase the local bearing resistance of the soft subgrade.

Design of geosynthetic-reinforced construction platforms is commonly based on local bearing capacity or on slope stability. Several researchers have suggested different bearing capacity factors, N_c, for unreinforced versus geotextile-reinforced and geogrid-reinforced unpaved platforms as covered in Chapter 9. A single layer of geosynthetic is commonly used.

Many ground modification techniques require the mobilization and operation of heavy equipment over the soft soils that are to be modified. Contractors often use a construction working platform for temporary access and equipment support. The design of construction working platforms is, therefore, often performed by the contractor.

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